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PROCEEDINGS OF THE DICE THROW SYMPOSIUM 21-23 JUNE 1977

Volume 3

General Electric Company—TEMPO
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July 1977

Proceedings

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Director
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Ground Motion	

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This report contains the proceedings of the DICE THROW Symposium held 21-23 June at the Ballistic Research Laboratory, Aberdeen Proving Ground, Maryland. The presentations contained herein describe the experiments conducted, instrumentation used, and results obtained by the various participating projects from detonation of the 600-ton ANFO HE test 6 October 1976 at the White Sands Missile Range, N.M. The DICE THROW Event was sponsored by the Defense Nuclear Agency.

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FOREWORD

This report contains the proceedings of the DICE THROW Symposium held 21-23 June 1977 at the Ballistic Research Laboratory (BRL), Aberdeen Proving Ground, Maryland. The report is divided into four volumes. Volumes 1 through 3 contain the unclassified presentations and Volume 4 contains the classified presentations.

The DICE THROW Event, which was conducted near the Giant Patriot site on the White Sands Missile Range (WSMR), 6 October 1976, was the final test of the DICE THROW Program. The charge for this test was composed of approximately 628 tons (570 metric tons) of ammonium nitrate fuel oil (ANFO). The charge configuration was a right-circular-cylinder base tangent to the surface with a hemispherical top, the same configuration as the second event in the Pre-DICE THROW II Series. The primary objectives of this test were to provide a simulated nuclear blast and shock environment for target response experiments that are vitally needed by the military services and defense agencies concerned with nuclear weapons effects, and to confirm empirical predictions and theoretical calculations for shock response of military structures, equipment, and weapon systems.

A complement of 33 experimenters and support agencies (including foreign governments) participated in Event DICE THROW. For details pertaining to the as-built experiment configurations, site and charge descriptions, and fielding requirements in support of this program, refer to the DICE THROW Test Execution Report, POR 6965.

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26. NORWEGIAN BLAST DOOR

by

Golden E. Lane, Jr.

Civil Engineering Research Facility

ABSTRACT

The Norwegian Defense Construction Service (NDCS) fielded an experiment in the DICE THROW Project. The NDCS experiment consisted of fabricating and exposing a concrete ammunition storage facility blast door to an overpressure of 1.4 MPa. The University of New Mexico's Civil Engineering Research Facility (CERF) was responsible for the construction, monitoring, instrumentation, and reporting of the experiment. The purpose of the experiment was to qualify the proposed blast door structure as a Norwegian, and possibly NATO, standard for ammunition storage facilities and civil defense protective installations. The door was instrumented with eight strain gages on the reinforcing steel, two accelerometers on the underside of the door, two accelerometers on the reaction frame of the door, and two active and four passive displacement gages to measure relative displacement between the door and the reaction base. The door survived the blast with a relatively small amount of permanent deformation and a moderate amount of cracking on the underside of the door.

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The Norwegian Defense Construction Service (NDCS) fielded an experiment in the
DICE THROW Project, a 600-ton, high-explosive test conducted at the White Sands
Missile Range in New Mexico on October 6, 1976. The NDCS experiment consisted of
fabricating a reinforced concrete blast door and companion reaction support and
exposing the unit to an overpressure of 1.4 MPa. The Defense Nuclear Agency's
Field Command supported the experiment and the University of New Mexico's Civil
Engineering Research Facility (CERF) was responsible for construction, monitoring,
instrumentation, and reporting of the experiment.

The purpose of the Norwegian experiment in Project DICE THROW was to qualify the
proposed blast door structure as a Norwegian, and possibly NATO, standard for
ammunition storage facilities and civil defense protective installations.

SECTION 2
EXPERIMENTAL PROGRAM

The blast door was constructed of reinforced concrete encased in a steel frame. The reaction pit was also reinforced concrete with a steel bearing angle to support the door. The details of the door and reaction pit are shown in figure 1. Figure 2 shows the location of the blast door in the DICE THROW testbed.

The steel frame, reinforcing steel, and bearing angle were fabricated in Norway and shipped to CERF as one unit (fig. 3). The only apparent damage to the door during shipment was one bent anchor (fig. 4). After the strain gages and instrumentation mounting plates had been installed and the concrete had been cast and cured at CERF, the door was transported to the White Sands Missile Range to be positioned in the reaction pit forms before casting of the reaction pit. Figures 5 and 6 show the placement of the door and the casting of the reaction pit, respectively. After the reaction pit concrete had cured, the door was opened to assure proper operation. Figure 7 shows the Norwegian experiment just prior to the test. The compressive strength of the concrete in the door was 43.6 and 49.7 MPa at 28 days and shot day, respectively, based on the average of two 152.4 x 304.8 mm cylinders for each date. The average 28-day compressive strength of the reaction pit was 37.8 MPa based on an average of five 152.4 x 304.8 mm cylinders.

Active instrumentation consisted of eight strain gages on the reinforcing steel in the door, four accelerometers--two on the door and two on the reaction pit, and two displacement gages under the door. In addition, there were four passive displacement measurements made with scratch gages between the bottom of the door and the floor of the reaction pit. The strain gages used were Micro-Measurement Type EA-06-500GC-350 which had a gage length of 12.7 mm, a nominal resistance of 350 ohms, and a gage factor of 2.095. Accelerometers were Endevco 2264-A-2KR and CEC Type 4-202-0001. The Endevco accelerometers were full bridge with two active piezoresistive strain gages. The CEC accelerometers were full bridge with four active piezoresistive strain gages. The active displacement transducers were Celesco Model PT-101-10 pull-wire potentiometers with special features to allow relative acceleration magnitudes of ± 5.08 m/sec with displacements of ± 127 mm.

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Figure 1. Blast Door and Reaction Pit (1 of 2)
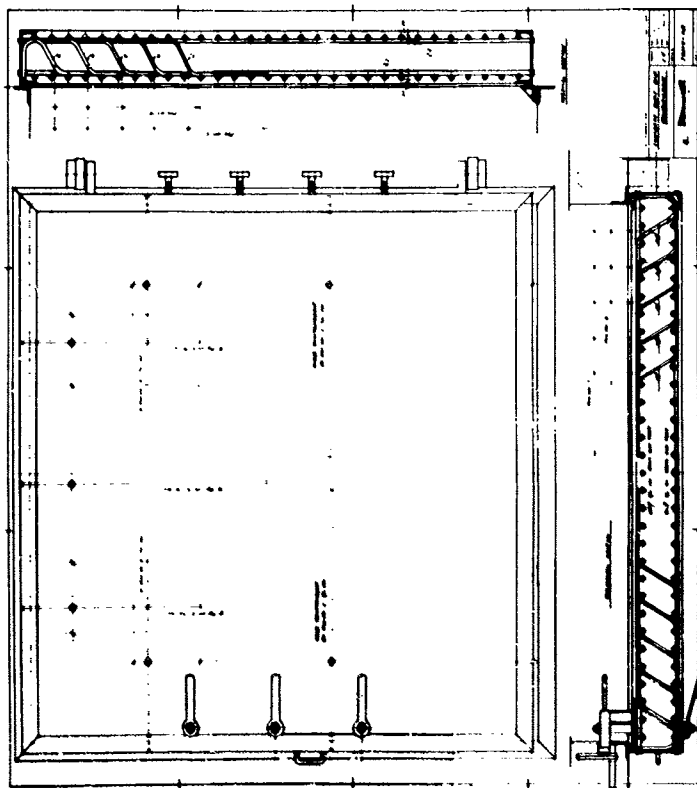
Figure 1. Blast Door and Reaction Pit (2 of 2)

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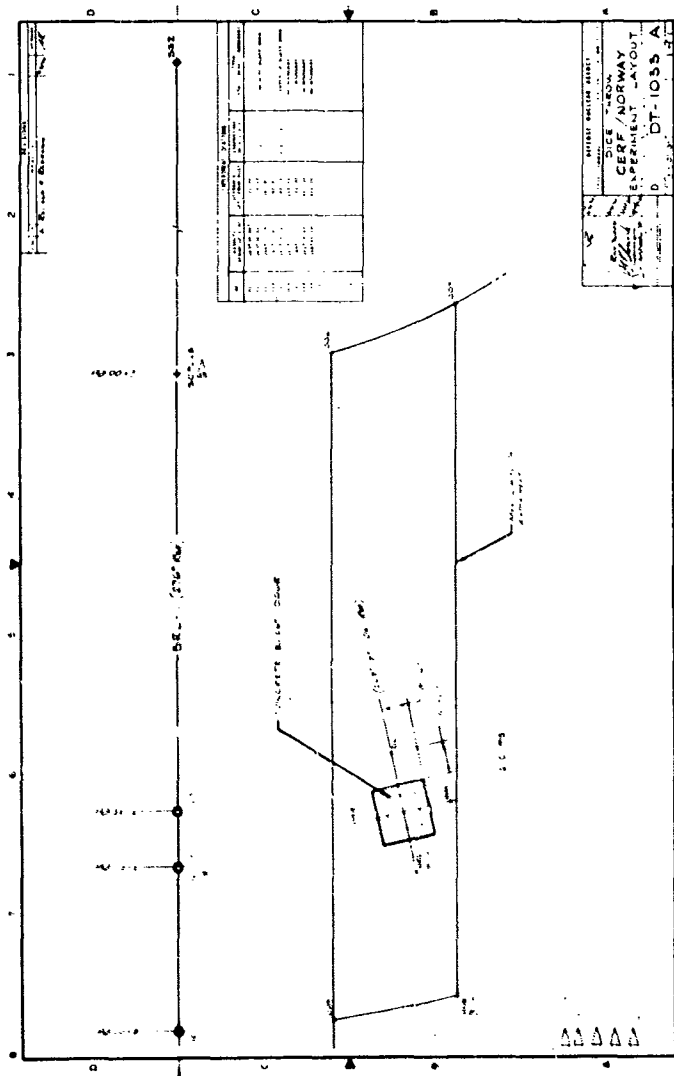
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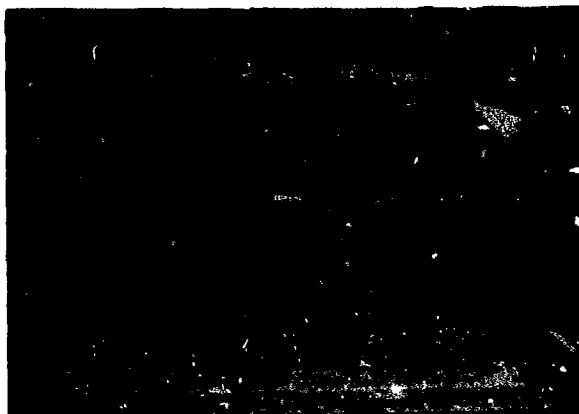
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Figure 4. Damaged Anchor

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Figure 5. Placement of Door
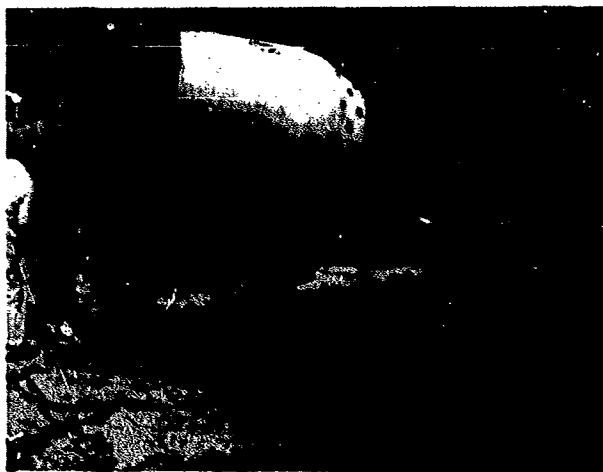
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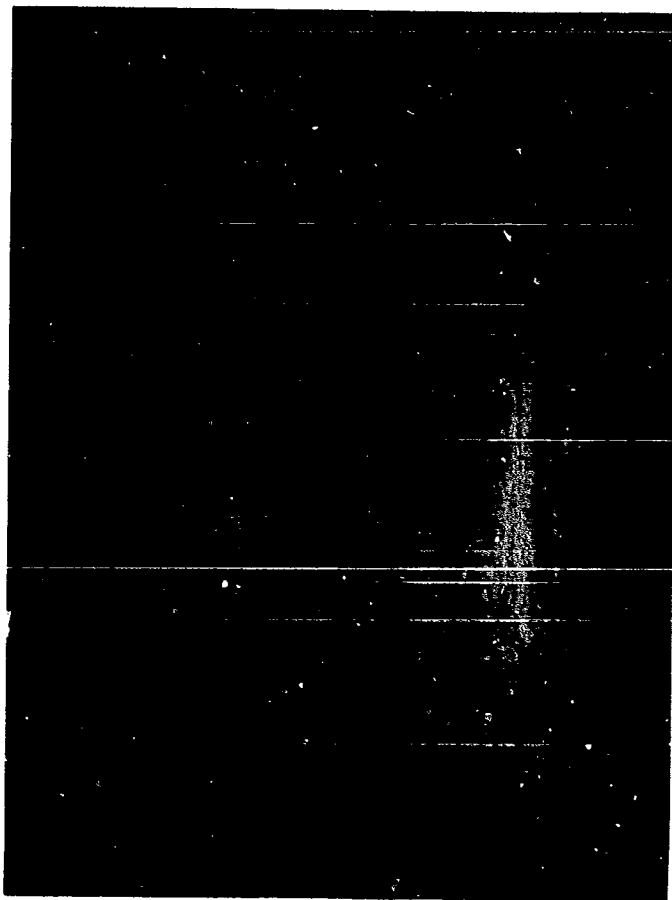
Figure 7. Preshot Norwegian Experiment

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The scratch gages used for the passive displacement measurements were simply two aluminum pipes, one inside the other, one attached to the door and the other attached to the floor of the reaction pit. Displacement was indicated by a pointed screw through the outside pipe scratching the inside pipe.

The active gages were connected to a steel junction box located approximately 300 m from the shelters with 4-conductor lead wire buried 1.2 m deep. The junction box was connected to the recording van by 20-pair cables. The recording van was approximately 1800 m from the junction box.

The recording van used for data acquisition was supplied by DNA (Van No. 36040). In the van, the bridge type transducers were excited and conditioned by B&F 1-171 Signal Conditioners. The conditioned signals were amplified with Bay Labs 5503 Amplifiers (dc - 50 kHz). Recording was accomplished on Sangamo Type 4784 32-Track Tape Decks. Wideband FM recording (108 kHz center with ± 40 percent deviation) was used.

In addition to data, IRIG-B time code and fiducial signals were recorded on each tape deck. During the event, the van was operated remotely from the timing and firing van.

After the event, quick-look data were played back on O-graph paper. Final copy data were prepared at the Air Force Weapons Laboratory data-reduction facility.

A sampling rate of 20,000 points per second and a filter frequency of 5 kHz were used in digitizing the analog data. Each channel was scaled in engineering units and plotted against time.

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SECTION 3
TEST RESULTS

Blast pressure was not measured at the exact location of the blast door. However, there was a blast pressure gage on gage line BRL-1 at the same radial distance from ground zero as the door. The relative location of the pressure measurement and the door can be seen in figure 2. The data from the pressure measurement are shown in figure 8.

Appendix A contains the time histories of the recorded data. Figure 1 and table 1 locate and describe the measurements, respectively. The sign convention used was as follows:

Acceleration	+ upward
Displacement	+ downward
Strain	+ compression

The displacements as indicated by the scratch gages were as follows:

Gage Designation	Displacement, mm
PD1	19.1
PD2	38.1
PD3	31.8
PD4	7.9

The door suffered some damage as can be seen in the posttest photographs in appendix B. After the test, the door was opened to determine the survivability of the hinges and locking mechanism. Although the door was opened with very little difficulty, it could not easily be reclosed because of distortions in the hinges.

PRESSURE (KPA)

1400.
1200.
1000.
800.
600.
400.
200.

14
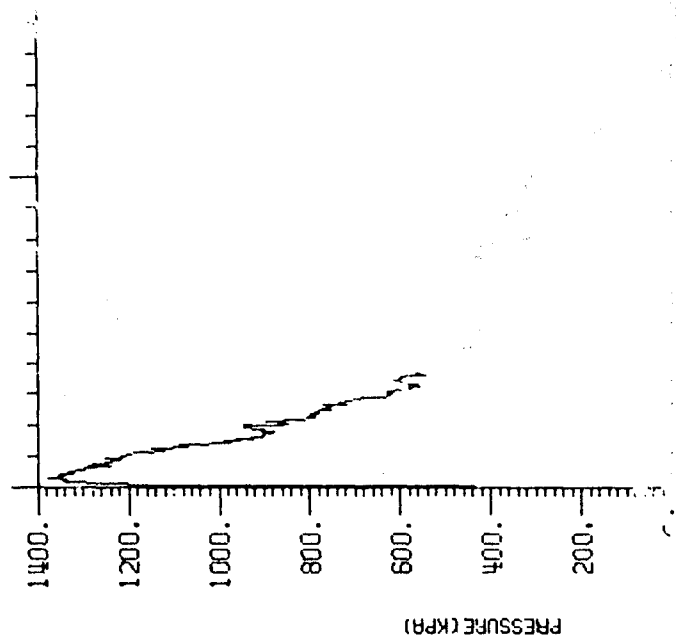
Table 1. Measurement List

Designation	Type	Location	Measurement Number
S1	Strain	Rebar - Top	13
S2	Strain	Rebar - Top	14
S3	Strain	Rebar - Top	15
S4	Strain	Rebar - Top	16
S5	Strain	Rebar - Bottom	17
S6	Strain	Rebar - Bottom	18
S7	Strain	Rebar - Bottom	19
S8	Strain	Rebar - Bottom	20
A1	Acceleration	Door - Center	21
A2	Acceleration	Door - Edge	22
A3	Acceleration	Reaction Pit - Vertical	23
A4	Acceleration	Reaction Pit - Horizontal	24
D1	Displacement	Door - Center	25
D2	Displacement	Door - Edge	26
PD1	Passive Displacement	Door - Off Center	
PD2	Passive Displacement	Door - Off Center	
PD3	Passive Displacement	Door - Center	
PD4	Passive Displacement	Door - Edge	

APPENDIX A

TEST DATA

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S2

SECONDS

STRAIN IN.1

SECONDS

STRAIN (UE)

S3

22

SECONDS

STRAIN, (μE)

S3

23

24

SECONDS

S4

STRAIN — (μ£)

SECONDS

STRAIN (UE)

400.
0
-400.
-800.
-1,200.
-1,600.
-2,000.
-2,400.

0 .1 .2 .3 .4 .5

S5

26

SECONDS

STRAIN (UE)

600.
0
-800.
-1200.
-1800.
-2400.
-3000.
-3600.
-4200.

0
.1
.2
.3
.4
.5

S6

S6

SECONDS

.20 .15 .10 .05 0

800. 0. -600. -1200. -1800. -2400. -3000. -3600. -4200.

STRAIN (UE)

31

Gage Broke

S8

STRAIN (in.)

SECONDS

.1000.
0
-1000.
-2000.
-3000.
-4000.
-5000.
-6000.

0   .05   .10   .15   .20

32

34

SECONDS

A2

36

Gage Broke

A3

SECONDS

ACCEL. (DB)

Gage Broke

A3

SECONDS

100.　　50.　　0　　-50.　　-100.

ACCEL., (G6)

38

Gage Broke

SECONDS

ACCEL. (G)

A4

41

SECONDS

42

Gage Broke

DISP. (INS)

SECONDS

.05 .10 .15 .20

D2

APPENDIX B

BLAST DOOR DAMAGE

Door After Shot

Damage to Locking Handles

Hinge Area
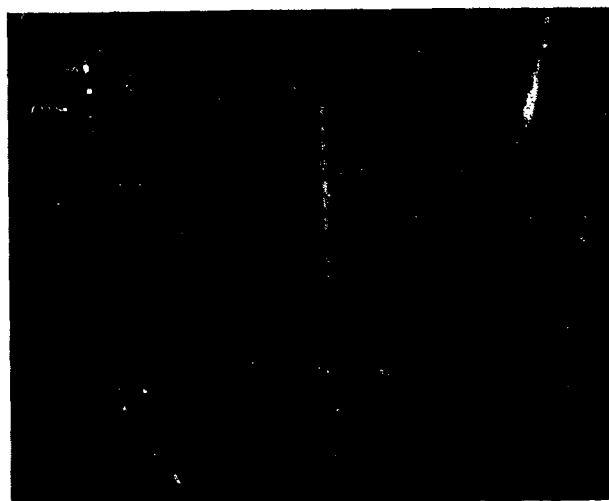
Damage to Hinge

Damage to Hinge

Top Surface Cracks

Top Surface Cracks
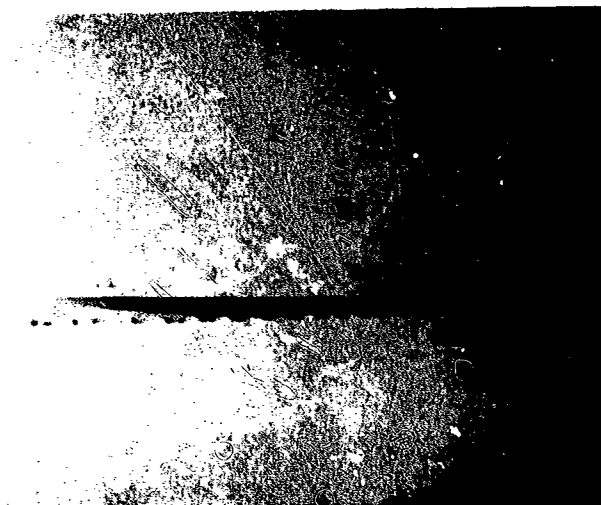
Damage to Seal Angle
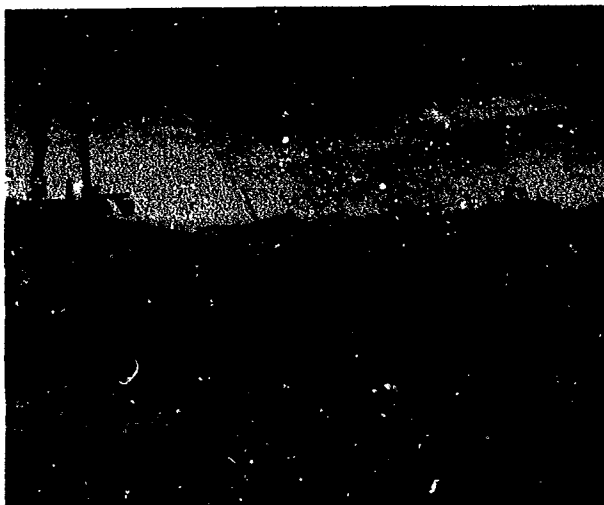
Lifting Door
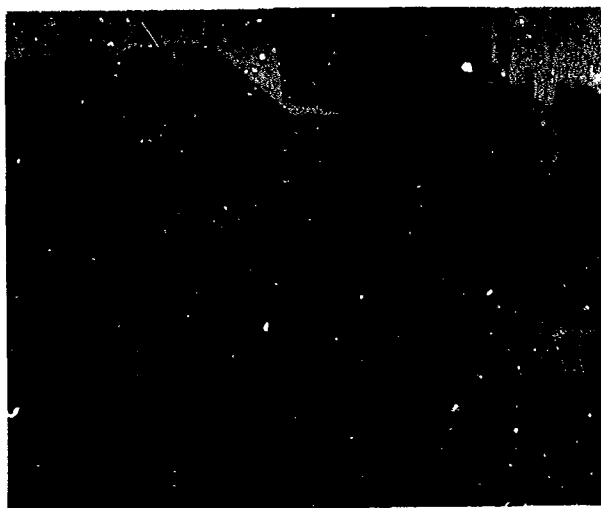
Permanent Set in Door

50

Door Crack Pattern
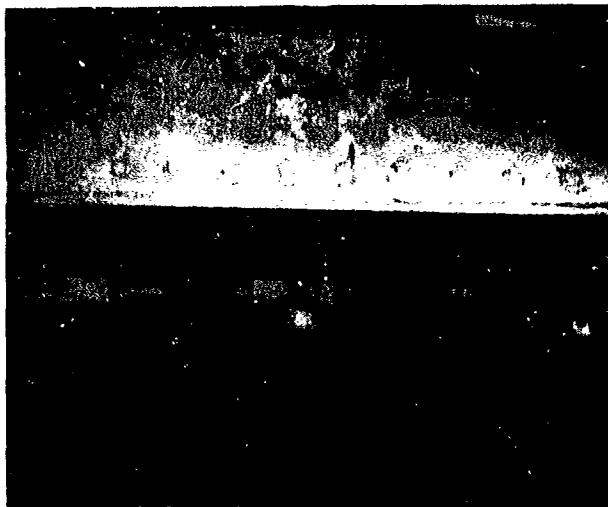
Deformed Lock Mechanism

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27. BLAST TESTS OF EXPEDIENT SHELTERS
IN THE DICE THROW EVENT

by

C.H. Kearny

Oak Ridge National Laboratory

BLAST TESTS OF EXPEDIENT SHELTERS IN THE DICE THROW EVENT[*]

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1. BACKGROUND AND SCOPE

Civil defense research at Oak Ridge National Laboratory (ORNL) has always stressed the development of protection against blast effects. One reason for this emphasis is the fact that, if a population is able to occupy shelters affording good protection against blast effects at overpressures up to 7 psi, the area in which serious blast casualties would be suffered would be reduced by at least 75% compared to the situation in which a population takes shelter in homes that would be badly damaged at the 2 psi overpressure range by the blast effects of a large weapon. Another reason is the fact that even expedient shelters, if their walls are skillfully shored and their entrances equipped with expedient blast doors, can readily be built so as to protect occupants against all blast effects at peak overpressure ranges several times as high as 7 psi. Therefore, in 1973 ORNL participated in Defense Nuclear Agency's (DNA's) MIXED COMPANY Event. This test subjected various expedient shelter designs to the effects of an explosion of 500 tons of TNT. All of the ORNL expedient shelters survived with little or no damage.[1] As a result, it was decided that the most promising designs should be subjected to blast effects severe enough to indicate the worst blast environments that these shelters are capable of withstanding. The main event of DNA's recent DICE THROW series afforded the required blast environment. This event was a 630-ton ANFO (ammonium nitrate-fuel oil) explosion, producing airblast effects about equivalent to a 1-kiloton nuclear surface burst. This shot was detonated on October 6, 1976 at White Sands Missile Range, New Mexico. Eighteen expedient shelters (including four half-scale models) were subjected to the blast effects

at overpressures ranging from 53 psi to 5.8 psi, and expedient life-
support equipment (mostly placed inside shelters) was exposed to over-
pressures of 53 psi to 1 psi. Several one-tenth-scale models of shel-
ters were also tested, at overpressures of up to 180 psi.

2. PRINCIPAL OBJECTIVES

The principal objectives of ORNL's participation in DICE THROW
were:

a. To obtain field data useful in making more reliable estimates
of the practical limitations of promising expedient shelter designs and
expedient life-support equipment, as regards their capabilities for
withstanding all blast effects from large explosions.

b. To observe the relative effectiveness of several different ways
of utilizing earth arching and trench-wall shoring to increase the blast
protection afforded by lightly constructed shelters, in order to develop
improved shelter designs that can be built using only widely available
materials.

3. INSTRUMENTATION USED AND TEST DATA RECOVERED

3.1 Blast Overpressures

Blast overpressures were measured by yielding foil membrane blast
gauges.[2] These passive gauges were developed at ORNL and performed well
at the lower overpressures (less than 7 psi). However, the ORNL gauges
that were installed adjacent to principal shelters to measure overpres-
sures above 7 psi all recorded overpressures 28-60% higher than those
recorded by the transducers at the same radial distances from ground
zero on DNA's adjacent Gauge Line No. 1. Therefore, we have used the
DNA measurements for all the aboveground overpressures to which the ORNL
shelters were subjected, except for the DNA measurement at the predicted
100-psi range, which was obviously far too low.

The distances from ground zero (GZ) to the shelters, the predicted overpressures, and the measured overpressures at these distances are shown in the table below.

Table 3.1. Overpressures at Various Distances

Distance from Ground Zero	Predicted Overpressures	Measured Overpressures
440 ft	100 psi	106 psi
540 ft	50 psi	53 psi
640 ft	30 psi	31 psi
740 ft	20 psi	20 psi
820 ft	15 psi	15 psi
1140 ft	7 psi	6.7 psi
1370 ft	5 psi	5.8 psi

To simplify this report, only a few references to distances from GZ or predicted overpressures will be made. Measured peak overpressures will be used (e.g., "53 psi," "31 psi").

The ORNL pressure gauges inside the shelters recorded low overpressures. All these gauges functioned well except those inside the shelters at the 31-psi overpressure range. All the ORNL pressure gauges were recovered, and the overpressures they recorded inside the shelters are used in this report.

3.2 Elastic and Permanent Deformations

Elastic and permanent deformations of the roofs and some other parts of the shelters were measured by passive mechanical devices.[1] Over 90% of these functioned effectively. Linear measurements of distances between parts of a shelter were taken before and after the blast.

4.

3.3 Blast-Wind Scouring

Blast-wind scouring of the earth mounded over shelters and around
entryways was determined by driving 12-in. steel spikes into the earth
until their heads were flush with the ground and measuring their expo-
sures after the blast. (The duration of blast winds is proportional to
the cube root of weapons yield;[3] thus the depth of scouring by larger
weapons can be estimated.) Also preblast and postblast depths of earth
over and around shelters were recorded.

3.4 Blast Damage to Structures

Blast damage to all structural parts of shelters and to the earth
walls of unshored shelters and of water-storage pits were determined
primarily by observation. Numerous photographs were taken, both before
and after the blast, to record blast damage--the most important part of
the test data.

4. SMALL-POLE SHELTER AT 53 PSI

4.1 Purpose

The Small-Pole Shelter (see Figs. 4.1 and 4.2) has been developed
for construction by unskilled workers in wooded areas (in stable or
unstable earth, below or above ground). It provides excellent protection
against radiation and much better protection against blast than does an
unshored trench shelter or any poorly shored shelter. Untrained groups
of families, using only muscle-powered tools, have succeeded in building
this type shelter in less than 48 hours elapsed time from the time they
received the instructions.[4] A 24-man section of an infantry platoon of
the 82nd Airborne Division, with no prior training and using only muscle-
powered tools, built a 24-man model, without benches or bunks, in 18
elapsed hours.[5]

4.2 Construction

The main room and the horizontal part of the entryway at the east
end were of unmodified Russian design,[6,7] except that the excavation in

Fig. 4.1. Plan and Elevation of Small-Pole Shelter.
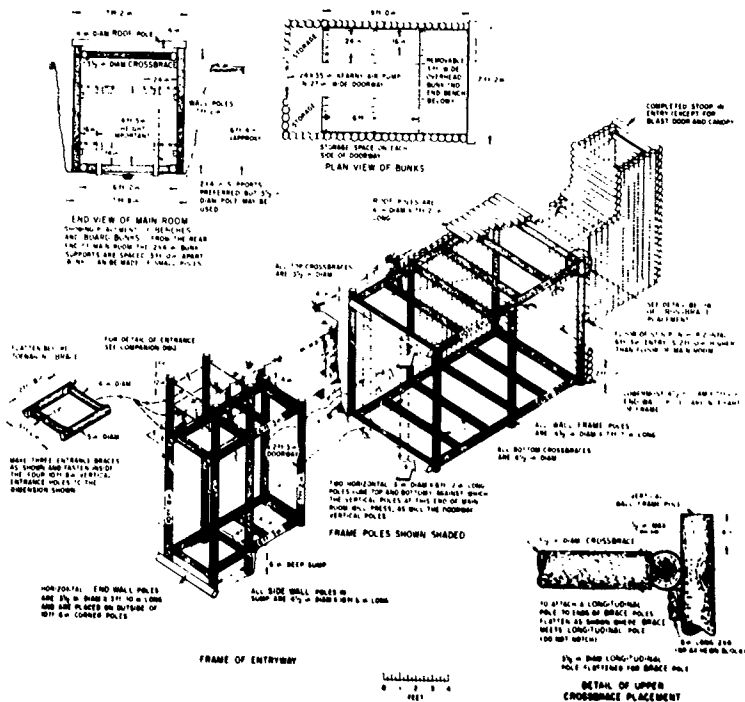
Fig. 4.2. Pictorial View of Small-Pole Shelter.

the hard caliche was made 2 ft deeper than the final level of the shelter floor. Then this bottom 2 ft was backfilled with dry, sandy earth. This soft earth under the wall poles permitted them to be pushed down sufficiently under blast loading to throw most of the load onto the resultant earth arching that blast overpressure sets up over a yielding structure.

A previous ORNL analysis[8] of the survivability of this shelter indicated that without the protection of earth arching it would withstand an overpressure from a 200-kiloton weapon of about 15 psi with blast doors closed. This analysis assumed the use of green hardwood poles, the strengths of which were determined in the ORNL materials laboratory. The roof poles and wall poles of all the ORNL pole shelters in DICE THROW were ponderosa pine. In this shelter, the poles averaged about 5 in. in diameter including their bark. The 12-occupant shelter room was 10-1/2 ft long, as illustrated by Figs. 4.1 and 4.2.

The horizontal part of the entryway at the south end was only 4-1/2 ft in height, with its floor 2-1/2 ft above the floor of the main room and the east-end entryway.[6,7] This height proved adequate, and this stoop-in entryway required significantly less material and labor to build than did the Russian-type horizontal entryway with 6 ft of headroom. (An unmodified Russian Small-Pole Shelter has only a small chimney-like air duct at one end; ORNL tests had proved that this small air duct would provide such inadequate ventilation that fatalities from excessive heat-humidity could result in warm or hot weather after a day of full occupancy.) The vertical entryways were of ORNL design,[7] as shown in Figs. 4.1 and 4.2, except that they extended 5 ft above the ceilings of the horizontal entryways. (The Russian inclined stairway-entrance had been found to be weak and not suitable for the installation of a blast door.)

The roof poles of this box-like shelter were at ground level. The length of this shelter was perpendicular to the radius from GZ. To provide adequate shielding against the initial nuclear radiation to be

expected at the approximately 50-psi overpressure range from smaller
nuclear weapons, the roofs of the shelter room and its entryways were
covered with 5 ft of mounded earth. For adequate protection against
initial radiation from a tactical weapon (through the entries), each
entryway should have been at least 10 ft long. For protection against
radiation from strategic weapons, the entries actually built would be
satisfactory, and only 3 ft of earth cover would give a protection
factor (PF) of over 500.

The need for blast doors on family shelters has long been recog-
nized.[9,10] ORNL blast tests[1] had demonstrated the effectiveness of
expedient blast doors at overpressure ranges up to 29 psi, and since the
present Soviet nuclear arsenal could subject over half of all Americans,
if in their normal areas, to serious blast dangers, we included three
new designs of expedient blast doors in our DICE THROW tests.

Both entrances of the Small-Pole Shelter were protected by expedient
blast doors (see Figs. 4.3 and 4.4). Each door measured 48 in. x 42 in.

PHOTO
6543-76

**Fig. 4.3. Nailing Tire-Strip Hinges to Expedient Blast
Door Tested at the 53-psi Overpressure Range.**
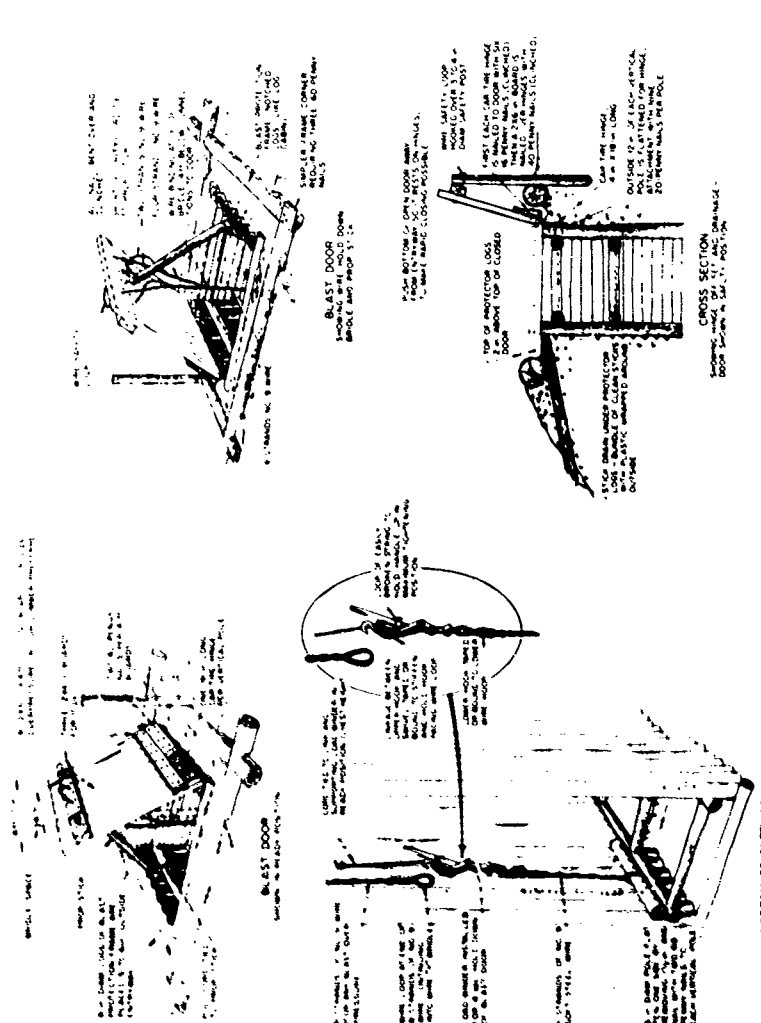
Fig. 4.4. Expedient Blast Door that Can Be Closed and Secured in Four Seconds. The expedient blast doors tested at DICE THROW were all of stronger construction than this door made of boards. This door was damaged but sill intact after being subjected to about 17 psi in the main explosion of DNA's MIXED COMPANY EVENT.

and each was made of five thicknesses of 3/4-in. exterior plywood. The plywood sheets were glued together with waterproof resin and nailed together from both sides. Expedient hinges made of strips cut from the worn treads of automobile tires were nailed to the door and to vertical poles of the entry with No. 16 (4-in.) coated nails, on a rectangular spacing of 4 inches in each direction. A door was hinged on its side nearest ground zero with 5 hinges nailed to the 5 vertical poles of this side of its vertical entry. Each hinge was a 24-in.-long strip of worn, wide-tread automobile tire, 4 to 6-1/2 in. wide and 1/4- to 1/2-in. thick, measured in the grooves of the tread. Each strip was nailed to its door with twelve 5-in. nails, driven in about 3-1/2 in. with their heads bent away from the hinge line.

After seeing the bright light from a nuclear explosion, an alert shelter occupant can close and secure this type door within 4 seconds. This is fast enough to effect the closure of the door before the arrival of the airblast shock wave from an 8-MT or larger weapon at the 20-psi or less overpressure range, but not fast enough at the 53-psi range. (See Fig. 4.4.) Therefore, if this shelter is to afford protection against tactical weapons, it should be equipped with expedient blast valves of the tire-strip type, installed in separate intake and exhaust shafts. This type valve has been blasttested without being damaged at 65 psi.[1]

Each blast door was surrounded with blast-protector logs which had been notched and spiked together and were evenly spaced around the door. These logs (about 8 in. in diameter and 8 ft long) had been placed with their upper sides about 2 in. higher than the top of the closed blast door.

4.3 Test Results

Figure 4.5 shows the four blast-protector logs around the north-end door after the blast. This explosion produced a measured peak overpressure

PHOTO
6545-76

Fig. 4.5. Blast-Protector Logs around Blast Door after
Being Moved by Blast Effects at the 5 -psi Overpressure Range.

of about 53-psi and a calculated peak blast-wind velocity of about 1,000
mph at this range (i.e., 540 ft from ground zero). The blast winds blew
away up to 12 inches of the dry earth previously piled around the blast-
protector logs. The shock wave and dynamic drag effects shifted these
four logs from their original positions. In its final position, the log
nearest ground zero was so close to the hinges that the door could be
opened from the inside to an inclination of only about 60 degrees.

If this door and its protector logs had been subjected to the same
overpressure from a large surface burst that would have produced dynamic
drag and blast wind effects of much longer duration, the door might have
been jammed in its closed position by the shifted logs. If long, strong
stakes had been driven prior to the blast so as to secure the logs,
their movement would have been reduced. However, for maximum blast
protection against nuclear weapons this whole shelter should have been
positioned deep enough in the earth so that its blast doors would have

been only a few inches above ground level, with the earth surrounding
the blast-protector logs sloped up around them at an angle of less than
10 degrees. Or the earth mounded over the whole shelter should have all
its slopes less than 10 degrees.

The pole frame and plywood blast doors of the Small-Pole Shelter
were essentially undamaged by the blast effects at the 53-psi over-
pressure range (see Fig. 4.6). However, occupants would have been
injured if they had been standing with their heads close to the ceiling,

PHOTO
0704-77

Fig. 4.6. Small-Pole Shelter after Being Tested with Blast
Doors Closed at the 53-psi Overpressure Range. Note the slightly
damaged expedient shelter-ventilating pump in the stoop-in entryway.
Two men worked about 5 min. to replace the 4 blown-loose flaps, the
only damage.

which was rapidly depressed when pressure on the roof poles caused the
wall poles to be punched down into the soft, back-filled earth supporting
them. This downward movement of the roof and walls varied from a minimum
of 2 in. in the southwest corner to a maximum of 6-1/4 in. in the north-
east corner. Figure 4.7 shows the movement at the center of the room, where

13.

Fig. 4.7. Movement of Upper Part of Small-Pole Shelter
Away from Ground Zero due to Blast Effects at 53-psi Over-
pressure Range.

the upper part of the shelter was moved 4-1/4 in. away from ground zero
and 4-3/16 in. downward, relative to the "fixed" vertical post to which
the lower part of the damaged deflection gauge was attached. Further-
more, about 15% of the floor area "puffed up" from 2 to 8 in. above its
original elevation.

Figure 4.8 shows how the floor "puffed up" about 6 in. in the
northeast corner of the shelter in the east entryway; some pressurized
earth caused some earth to "flow" up into the closed room, in which the
measured peak overpressure was only 1.5 psi. About 85% of the floor
area was undisturbed, as was the floor in front of the man's hand resting
on the cross brace. Neither the blast gauge resting on the brace pole
in the corner nor the small expedient fallout meter on top of it was
moved.

PHOTO
6459-76

Fig. 4.8. "Puffed-Up" Part of the Floor of Small-Pole
Shelter due to the Start of Earth Flow under Moderately Long-
Duration Blast Overpressure at the 53-psi Overpressure Range.

If a person had been standing on the floor when it was "puffed up"
suddenly, possibly his legs could have been injured. To prevent possible
injuries due to an intact ceiling moving very rapidly downward and/or
the floor moving upward, occupants could recline in expedient bedsheet
hammocks[11] slung from the upper horizontal brace poles of the main
shelter room, as shown in Fig. 4.9.

PHOTO
6458-76

Fig. 4.9. Expedient Bedsheet Hammock, Useful to Avoid
Sever Shock Effects in a Shelter at High Overpressure Ranges.
The man is operating an expedient shelter-ventilating pump via
an expedient pulley-equivalent, a greased forked stick suspended
on strings.

The whole roof, the upper horizontal braces, and the upper ends of
the wall poles were all displaced about 4-1/4 in. to the west (away from
GZ) by the blast effects on the 5-ft-high mound of shielding earth over
the shelter. The sides of this mound sloped about 36 degrees; its width
on top averaged about 10 ft. (If this dry mound had been subjected to
the blast effects of a megaton or larger nuclear weapon at the same 53-
psi overpressure range, the much greater impulse and longer-duration
drag effects might have caused the earth mound to be displaced far
enough to wreck the underlying pole shelter--especially since the long-
duration blast-winds would have scoured away most of the earth cover.
Even a mound of wet earth, which is much less vulnerable to long-duration
blast-wind scouring, might have been displaced far enough to cause
serious or disastrous structural damage.)

The maximum overpressure measured inside the shelter was 1.5 psi--
not enough to be harmful. Less than half of this pressure increase was
due to the sudden reduction in the volume of the shelter room which was
described above. The rest was caused by blast wind that blew through
cracks between the poles near the top of the vertical entryways. These

cracks appeared after the initial blast wind had scoured away several
inches of the covering earth and torn away the polyethylene film that,
with the essential help of small-scale earth arching, had kept earth
from being forced between the cracks by the peak overpressure.

There was no damage to any of the life-support equipment in this
shelter, except for quickly repairable damage to the expedient shelter-
ventilating pump (KAP)[12] pictured in Fig. 4.6.

4.4 Conclusions and Recommendations

4.4.1 - A Small-Pole Shelter built in stable ground and equipped
with blast doors can give reliable protection against the blast effects
of small tactical weapons up to about the 50-psi overpressure range.

4.4.2 - A modification of this shelter with a continuous pole floor
under the wall poles should not fail as a possible result of a large
amount of pressurized and destabilized earth flowing up into it through
its floor when subjected to the long-duration overpressures and large
movements caused by a megaton explosion.

4.4.3 - In order to prevent the above modification from seriously
reducing the capability of the shelter frame to yield under blast loading
and thus promote protective earth arching, all parts of one of the
shelters to be tested should first be covered with readily crushable
material, such as small tree limbs. Then this material should be covered
with fabric or plastic before placing earth around and over the protected
shelter.

4.4.4 - Small-Pole Shelters modified in these ways should be sub-
jected to the effects of blast simulating at least a 100-KT explosion
at the 25-psi and 50-psi overpressure ranges, when installed in a trench
dug in unstable earth, deep enough so that its blast doors are only about
a foot above the original ground level.

5. UNMODIFIED RUSSIAN POLE-COVERED TRENCH SHELTERS AT 20 AND 6.7 PSI

5.1 Purpose

Two identical unmodified Russian Pole-Covered Trench Shelters were tested at the 6.7-psi and 20-psi overpressure ranges, in order to make a more accurate estimate of the blast protection afforded occupants of this common type of Russian expedient shelter. This unshored "dugout" is recommended for construction in stable earth.

5.2 Construction

The two unmodified Russian Pole-Covered Trench Shelters were of the design detailed in the 1969 Soviet civil defense handbook[13] except that the entrance stairways were at right angles to their lengths, a modification recommended in both the 1972 and the 1976 Russian shelter-building manuals.[14,15] Figure 5.1 shows most of the roof poles in position before the shelter was covered with 4-mil polyethylene and earth mounded 30 in. deep. A total of 62 lodgepole pine poles, each 7 ft long, were laid side by side across the 31-ft-long trench

PHOTO
6406-76

Fig. 5.1. Poles Covering Russian Pole-Covered Trench Shelter at 20-psi Overpressure Range, with Uncompleted Stairway Opening Facing Away from Ground Zero.

18.

(not including the right-angle entry stairway that is shown in the
foreground of Fig. 5.1). Figure 5.2 gives the details of this simple
fallout shelter.

5.3 Location and Test Results

A Soviet civil defense handbook[6] states that within "the zone of
complete destruction" the overpressure exceeds 0.5 kg/cm^2 ($\sim$ 7 psi) and
that all residential and industrial buildings and all fallout shelters
will be destroyed. (This limitation obviously does not apply to the
Russian "hasty shelters" built of prefabricated concrete or steel com-
ponents. Typical Russian expedient fallout shelters are of light
construction and are not designed to withstand blast effects.) There-
fore, one unmodified Russian Pole-Covered Trench Shelter was built at
the forecast 7-psi overpressure range (6.7 psi was measured). Because
of the almost rock-like caliche earth, an identical shelter was built at
the 20-psi range, to see if occupants might survive more severe blast
effects than those at the 7-psi range. Neither shelter had a blast
door.

In the shelter at 20 psi, two anthropomorphic dummies (supplied by
the Lovelace Foundation) were seated side by side just inside the inner
curtain (see Figs. 5.2 and 5.3). A movie camera was installed by Denver
Research Institute for the U.S. Army's Ballistic Research Laboratory.
This camera was farther inside the shelter, mounted on a concreted-in-
the-ground post. This camera took 400 frames per second; the four
photographs of Fig. 5.3 were taken in 1/100 second. The first photograph
shows only a slight movement of the innermost blanket-curtain. The
second shows the earth walls beginning to crumble under the forces of a
ground shock wave, induced by the airwave slap overhead before the
airborne shock wave reached these walls or the dummies. The third and
fourth photographs show the innermost blanket-curtain being torn, revealing
the torn outermost curtain, that was darker colored, being blown behind
and against it. The collapsing walls trapped the two dummies before

Fig. S.2. Russian Pole-Covered Trench Shelter in Stable Earth

PHOTO 6632-76 (No. 1)

PHOTO 6629-76 (No. 3)

PHOTO 6636-76 (No. 2)

PHOTO 6634-76 (No. 4)

Fig. 5.3. Dummies Being Struck by Airblast and Curtains
Traveling about 180 mph. Note the walls collapsing under ground-
shock stresses before the arrival of the airborne shock wave.

the entering blast wind, which was shown by the four movie frames to
have a velocity of about 180 mph, could blow them over. (The blast wind
peaked at about 470 mph outside this shelter.)

Figure 5.4 shows the dummies trapped by the collapsing walls.
Because their strong steel joints did not permit these dummies to bend
forward, the collapsing walls did not bend them forward, knock them
down, and bury them, as would have been the fate of two men. Note the
unbroken roof poles.

PHOTO
0705-77

Fig. 5.4. Dummies at 20-psi Range after Ground Shock
Collapsed the Earth Walls of Shelter. Their steel "bones" and
joints prevented them from being knocked down and buried.

The measured overpressure inside this shelter was 7 psi--high
enough to break some persons' eardrums. (If this shelter had been sub-
jected to the blast effects of a megaton weapon at the 20-psi range, the
maximum overpressure inside the shelter would have been almost 20 psi.)

The entry was wrecked and much of its covering earth was blown
away, as illustrated by Fig. 5.5. The ventilation duct was broken off.

PHOTO
6460-76

Fig. 5.5. **Wrecked Entry of Russian Pole-Covered** Trench
Shelter at 20 psi.

At the 6.7-psi range, an identical shelter suffered serious damage.
Chunks of hard caliche weighing up to about 400 lb were broken off the
very stable earth walls and would have injured shelter occupants. A
dummy seated on a fixed bench next to the blanket-curtains was knocked
off the bench by the shock wave and the entering blast winds (see Fig.
5.6).

PHOTO
6462-76

Fig. 5.6. **Dummy Knocked Off Bench in Russian Pole**
Covered Trench Shelter at the 6.7-psi Overpressure Range.

5.4 Conclusions

5.4.1 - In soils typical of most inhabited areas, if a shelter of this design were subjected to the blast effects of a much larger explosion at the 7-psi overpressure range, the Russian estimate of "total destruction" would probably prove to be realistic. (The author believes that "total destruction" in this sense means the shelter would be so badly damaged as to be uninhabitable--not that all occupants would be promptly killed.)

5.4.2 - Earth arching in adequately thick earth cover over pole roofs prevents the poles from being broken by overpressures far in excess of the pressures such roofs could withstand if uncovered.

5.4.3 - Stresses due to ground shocks and earth waves would be the predominant causes of failure of unshored trench shelters subjected to the blast effects of large explosions.

6. LOG-COVERED TRENCH SHELTER AT 53 PSI

6.1 Purpose

We constructed an unshored trench shelter with its roof poles positioned in two different ways and located at the predicted 50-psi overpressure range because:

a. We anticipated that the extremely stable, rock-like caliche earth at the test site would result in unshored trench walls being so strong that they would not collapse under the ground shock stresses produced at the 50-psi range by mere 1-KT blast effects.

b. We were confident that effective earth arching in the thick earth would prevent the breaking of roof poles.

c. We were interested in comparing the effectiveness of the Russian and the Chinese way of roofing a trench with poles or logs.

6.2 Construction

This shelter was built with half of its 12-ft-long room having its roof poles positioned in the Russian manner at ground level (see Figs. 5.1, 5.2, and 6.1). The other half of the room had its roof poles positioned

in a recommended Chinese manner[16] (i.e., about 28 in. below ground
level). Figure 6.1 shows the vertical cross sections of these two
halves as modified from the original designs in order to permit a better
comparison between the merits of the two different ways of positioning
roof poles. (The room of the Russian half was made 16 in. less in
height than in the original Russian design, and the Chinese half was
made 4 in. less in width than specified in the Chinese handbook.[16])

PHOTO
DWG 77-10330
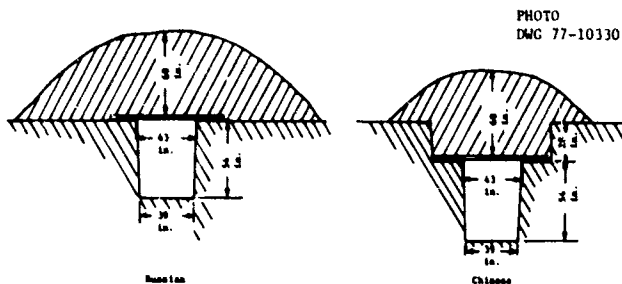
Fig. 6.1. Comparison of Russian Way and Chinese Way of
Positioning Poles to Roof a Trench Shelter. Note that the Chinese
way requires about 35% less earth to be moved in order to make a
5-ft-thick covering--about the thickness specified in a Chinese
handbook for shielding against initial nuclear radiation.

As shown in Fig. 6.1, the Chinese half was built with its roof
poles resting on earth shelves 28 in. below ground level, cut into the
hard caliche. All roof poles (logs) were ponderosa pine. The poles
averaged about 5 in. in diameter, not including their bark. All were
cut 7 ft long. Earth was mounded about 5 ft above ground level over
this whole shelter. This resulted in about 4-1/2 ft of earth covering
the roof logs of the Russian half and about 6 ft covering the roof logs
of the Chinese half. Blast-wind scouring removed a foot of this mounded
earth.

The vertical parts of the two entries to the shelter were of a newly developed design with triangular cross sections. The expedient blast doors were of a new triangular type. This design (see Figs. 6.2 and 6.3) was developed in order to: (a) use green poles cut from ordinary trees to make a tight-closing expedient blast door that takes advantage of the fact that three intersecting straight lines determine a plane, (b) require only widely available hand tools and common materials (e.g., auto tires, nails, and some wire or rope, in addition to poles), and (c) make practical the use of ʌ triangular vertical shelter entry, which has a smaller cross-sectional area than does a rectangular vertical entry big enough for the same sized person to use, and shows promise of requiring less materials to meet a given level of blast protection.

PHOTO
0700-77

Fig. 6.2. Expedient Triangular Blast Door Made of Pine Poles. The auto-tire flap valves over the 1-1/2-in.-wide spaces between the poles were undamaged by the blast effects at the 53-psi over-pressure range. Ground zero was to the left. Blast effects had moved the three connected blast-protector logs, preventing the door from being opened fully.

PHOTO
6442-76

Fig. 6.3. View of the Same Triangular Blast Door, Looking
Toward Ground Zero. The hinge pole, originally 7-in. in diameter
after peeling, had been flattened on its top and back side. The
two other poles, 8-in.-diameter, had been flattened on their bot-
tom, top, and inner sides. All three poles were notched and nailed
together. Note the slots between the door-covering poles.

6.3 Test Results

Figures 6.2 and 6.3 show the triangular blast door on the south end
of the shelter, undamaged after the blast. Note that one of the 3
blast-protector logs (the log in the lower left corner of the photograph)
has been pushed by the blast up against the hinge-pole of the blast
door. The door was undamaged. However, the movement of the earth mound
had broken the door-seat pole on which the man's foot is shown resting
in Fig. 6.3. Figure 6.4 shows the break more clearly.

Both of the triangular blast doors were undamaged. The expedient
blast valves on the blast doors were closed by the blast, and about 75%
opened after the blast, permitting adequate ventilation with an expedient
pump, a KAP. The overpressure inside the Chinese half was 1.5 psi, and
the overpressure directly under the north door was 3 psi. The results
of this test indicate that the use of expedient blast valves over the

PHOTO
6443-76

Fig. 6.4. Broken Pole of Triangular Door-Seat. This pole
was broken by differential earth movements of the earth mounded
over the shelter. The man's hand rested on the unbroken hinge-
pole.

1-1/2-in.-wide cracks of this blast door is impractical. Most of the
flap valves opened before the strong blast afterwinds subsided. These
winds plus the natural desert winds blew so much dirt and sand through
the valves and into the shelter that a serious fallout entry problem
could exist after a nuclear blast. Figure 6.5 shows the blast door at
the north end of the shelter before it was opened after the blast. Much
earth and sand had been deposited on it by the subsiding blast winds.

PHOTO
6441-76

Fig. 6.5. Posttest Condition of Expedient Triangular
Blast Door. Some flap valves had been jammed shut, and
much earth and sand had been deposited.

Although not one roof pole of any part of this shelter or any other
shelter was broken or cracked, the ground-shock effects collapsed the
walls of the Russian half of this shelter so badly (see Fig. 6.6) that
all occupants would have been killed. Damage to the Chinese half was

PHOTO
6445-76

Fig. 6.6. Postshot View of the Caved-In Caliche Walls of
the "Russian" Half of the Log-Covered Trench Shelter at 53 psi.

much less serious, although hundreds of pounds of caliche, some chunks
weighing up to 20 lbs, were broken off the edges of the shelves supporting
the roof logs. The roof-deflection gauge in the Chinese half recorded a
maximum transient downward deflection of 1-1/2 in. and a permanent
deflection of 7/8 in.

6.4 Conclusions and Recommendations

6.4.1 - Under the longer-duration blast effects of a large nuclear
explosion, mounded entries extending several feet above original grade
level would probably be wrecked by the combined effects of blast-wind
scouring and dynamic drag.

6.4.2 - Blast doors should be positioned only about a foot above
ground level, and earth should be mounded with slopes of 10 degrees or
less. (Unfortunately, such deeper excavation, even in softer earth,
might make construction within 48 hrs impractical for builders having
only hand tools.)

6.4.3 - Triangular blast doors made of poles can readily be built
to withstand 50-psi blast effects, but should be made solid and as
nearly dust-tight as practical. Separate ventilation shafts with blast
valves should be provided, with the blast valves positioned about 2 ft
from the bottom of each shaft.

6.4.4 - Persons building expedient shelters to provide protection
against nuclear blast effects should build well-shored shelters with
blast doors whenever practical.

7. LOG-COVERED TRENCH SHELTER AT 31 PS'

7.1 Purpose

A near-counterpart of the Log-Covered Trench Shelter that was
tested at 53 psi was tested at 31 psi, in order to determine at what
overpressure range this type shelter, if built in extremely stable
earth, will survive. Also we wished to test a semiexpedient design of
steel blast door on a shelter entrance at approximately 30 psi.

7.2 Construction

This shelter was constructed the same as the Log-Covered Trench
Shelter at 53 psi, except that protecting its single entry it had a
semiexpedient blast door made of about 65% of a 30-gal. steel oil drum.
Rubber-tire hinges and rubber-tire seals made a snug closure between the
door and the upper part of the vertical entry. The upper 2 ft of the
vertical entry was made of two thicknesses of 2-in. boards nailed together
(see Fig. 7.1).

7.3 Test Results

Although the blast effects loosened some of the bolts of the steel
blast door, tore the metal in several places, and produced other damage
indicating that it was on the verge of failure, it did not fail.

PHOTO
6493-76

Fig. 7.1. Semiexpedient Blast Door Made of a 30-gal Steel
Drum, Badly Damaged at 31 psi but Still Blast-Tight. Blast-wind
scouring had removed up to 17 in. of the dry earth mounded around
this entrance and blown away its single blast-protector log.

Figure 7.2 pictures the interior of the Chinese half of the shelter
after the blast had broken hundreds of pounds of caliche off the very
stable walls and lowered the roof but did not cause it to collapse. No
poles were cracked in any part of this shelter. The walls of the Russian
half collapsed so badly that all occupants would have been buried.
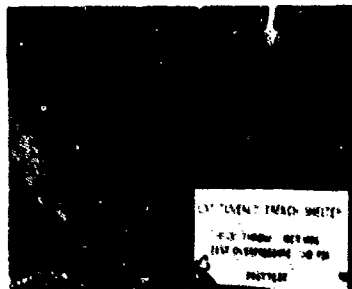
PHOTO
6446-76

Fig. 7.2. Serious Wall Caving at 31 psi (Predicted 30 psi).
The beam deflection gauge on top of the post showed a 2-1/2 in.
lowering of the center roof log.

7.4 Conclusion and Recommendations

7.4.1 - Even in extremely stable earth, an unshored trench shelter at 31 psi would give inadequate blast protection against even a small tactical nuclear weapon.

7.4.2 - The steel-drum blast door is not as blast resistant as pole or plywocd blast doors that require materials much less difficult to find and that require less skill, tools, and time to build.

8. DOOR-COVERED EARTH-ROLL SHELTERS AT 15 AND 5.8 PSI

8.1 Purpose

Two of these aboveground small fallout shelters,[7] made of interior hollow-core doors, bed sheets, and other materials available in tens of millions of American homes, were tested at the 15- and 5.8-psi overpressure ranges in order to determine if the shelters would afford better blast protection than would typical homes.

8.2 Construction

Each shelter was built with its long axis on a radius from ground zero. Figure 8.1 shows the interior of the shelter at the 15-psi range
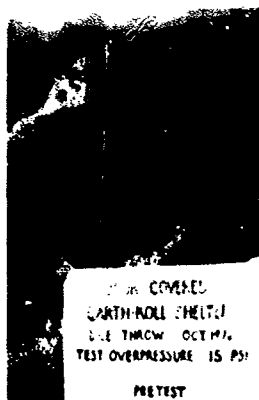
PHOTO
6497-76

Fig. 8.1. The Bedsheet "Earth-Roll" Walls Were 36 Inches Apart before Test.

before the explosion. The vertical stick touching a roofing door is a roof-deflection gauge, with its upper end consisting of nothing but a thin cylinder of household aluminum foil, an unsatisfactory device if exposed to blast wind. Figure 8.2 gives details of the construction of these shelters (see p. 33).

8.3 Test Results

Figure 8.3 is a posttest picture of the northward-facing entry of the shelter at the 15-psi overpressure range. This photograph also shows part of the northward-facing side of this shelter. The blast winds scoured only about 1 in. of earth from the top of this shelter, apparently because its long, flat top extended in the same direction that the blast winds blew. Note the proof of the toughness of polyester-cotton pillowcases used to make 100-lb sandbags. The sandbag in the foreground was blown about 7 ft by the approximately 370 mph blast wind without being broken.

PHOTO
6483-76

Fig. 8.3. Northward-Facing Entry (at Right Angle to the Direction to Ground Zero) of Door-Covered Earth-Roll Shelter at the 15-psi Overpressure Range. This is a post-test photograph.

To the surise of most observers, earth arching above the roof doors prevented any of them from being broken in by the blast effects. The doors
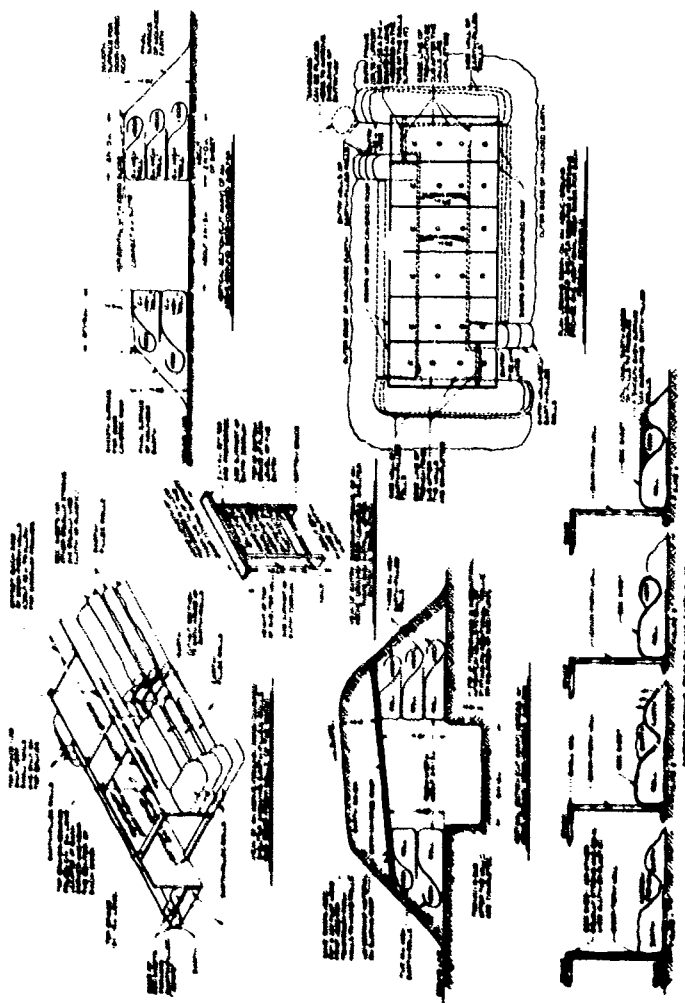
Fig. 8.2. Door-Covered Earth-Roll Shelter, Tested at 15 and 5.8 psi.

were not broken in, even though the lower 1/8-in.-plywood veneer of
three of the six doors was broken. Figure 8.4 pictures the interior of
the shelter at the 15-psi overpressure range after the blast effects
outside had caused the sandy soil inside the bedsheet "earth-rolls" to
"flow" inward rapidly. The width of the shelter was reduced from 36 in.
to a minimum of 14 in. No additional earth movements were observed
during the two weeks following this test. This unanticipated earth
within the "earth rolls" did not tear any of the pieces of bedsheet
cloth.

PHOTO
6484-76

Fig. 8.4. Interior of Door-Covered Earth-Roll Shelter
after 15-psi Blast Effects had Reduced Width of Shelter from
36 Inches to a Minimum of 14 Inches Near Its Center.

Pressure-time measurements on the adjacent DNA Gauge Line No. 1
showed that only about 40 msec elapsed between the peak overpressure of
14.9 psi recorded at the same distance (820 ft) from ground zero, and
its reduction to 6 psi, the maximum overpressure recorded inside

this shelter by the ORNL pressure gauge shown in the foreground of Fig. 8.4. The gauge that had been installed to measure the roof deflection was blown away by the entering shockwave and blast wind. The reduction in ceiling height appeared to be less than 1 in. in this part of the shelter, but up to about 4 in. in other parts.

At the 5.8-psi overpressure range, the Door-Covered Earth-Roll Shelter was still habitable for weeks after the test. Figure 8.5 shows that at 5.8 psi the walls were not forced inward by the blast effects. The unbroken upper 1/8-in. veneer plies of the doors held as flexible membranes, and earth arching was set up in time to prevent this shelter's roof from being collapsed either as a result of initial mechanized earth loading or due to the 5.8-psi blast effects.

PHOTO
6485-76

Fig. 8.5. Posttest Interior of the Door-Covered Earth-Roll Shelter at 5.8-psi Overpressure Range. The lower 1/8-in. veneer of the doors had been badly broken by impact before the test, due to a front-end loader having dumped tons of earth onto this yielding roof.

The peak overpressure measured inside this shelter was 3 psi, about half the 5.8 psi measured outside on DNA's adjacent Gauge Line No. 1. The blast winds, that peaked outside at about 175 mph, scoured away only a fraction of an inch of the shielding earth.

8.4 Conclusions and Recommendations

8.4.1 - A Door-Covered Earth-Roll Shelter obviously is impractical for use as a blast-protective shelter against blast effects considerably less than those at the 15-psi overpressure range from even a very small nuclear weapon.

8.4.2 - If this fallout shelter with a protection factor of at least 200 had been built in a typical suburb and had been subjected to the blast winds from a megaton weapon at the same 5.8-psi overpressure range, it might have been damaged or destroyed by blast-hurled pieces of houses and/or trees.

8.4.3 - Notwithstanding the hazards inherent in the use of this or any other lightly constructed aboveground shelter in a blast area, occupants of this simple shelter would have a decidedly better chance of surviving than would people inside typical suburban homes, which would be demolished by the blast effects at 5.8 psi.

9. RIDGE-POLE SHELTERS AT 15 AND 5.8 PSI

9.1 Purpose

In wooded areas having the water table or rock too close to the surface for belowground expedient shelters to be practical, untrained families with few tools have been able to build Ridge-Pole Shelters in less than 48 hours. No prior blast testing of this type shelter, that has its side poles merely leaning against its ridge pole, had been carried out anywhere (see Fig. 9.1).

PHOTO
6405-76

Fig. 9.1. Almost Completed Frame of Ridge-Pole Shelter
at 15 psi. Only the outermost roof pole of the entry has
been placed on its wall poles.

9.2 Construction

Two identical Ridge-Pole Shelters were built, each having the
dimensions shown in Fig. 9.2. One was tested at the measured 15-psi
overpressure range, and the other at the 5.8-psi overpressure range.
Each was positioned with its ridge pole perpendicular to a radius from
GZ, with one of its two crawl-in entries facing GZ and the other entry
facing in the opposite direction. Figure 9.1 shows the almost completed
pole frame, plus a temporary brace pole steadying the entrance. The
pole frame was next covered with small, leafy limbs (Fig. 9.3), which in
turn were covered with 4-mil polyethylene. Then a covering of dry,
sandy earth 2 ft thick was placed over the whole shelter, with earth-
filled potato bags retaining the earth over the entrances.

9.3 Test Results

9.3.1 - At 15 psi.--Contrary to our expectations that the blast
effects would collapse the main room, the main room was undamaged (see
Fig. 9.4). The ridge-pole was moved only one-half inch away from GZ.
However, up to 9 in. of earth was scoured off the top of the shelter.
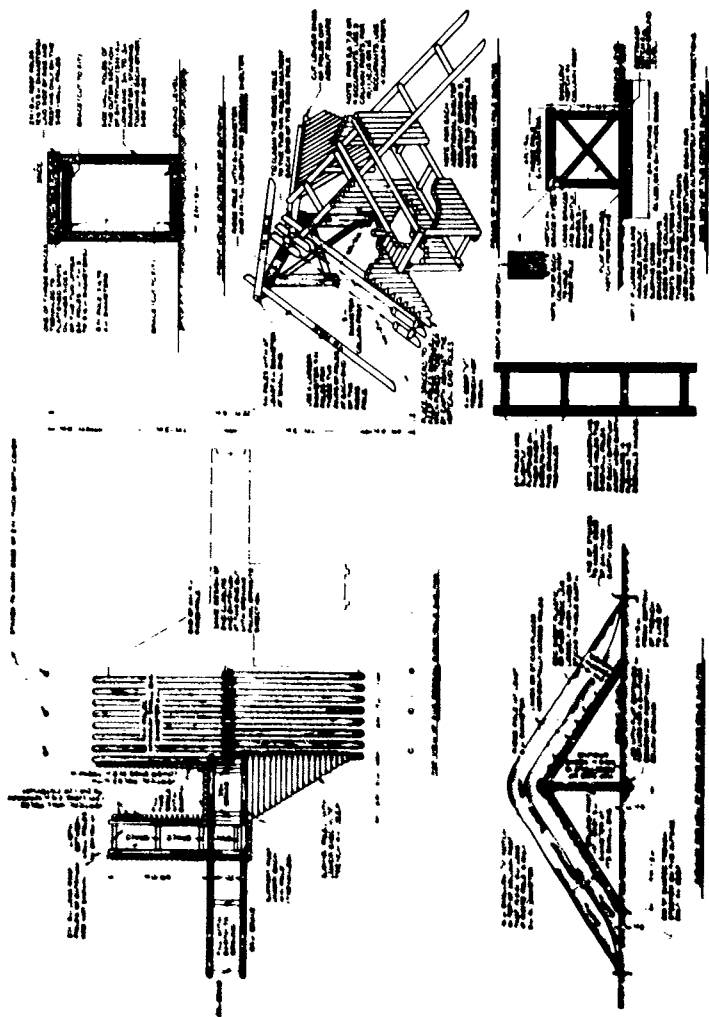
Fig. 9.2. Ridge-Pole Shelter Tested at 15 and 5.8 psi.

Fig. 9.3. Covering the Frame of a Ridge-Pole Shelter with Salt Cedar Limbs.

PHOTO
6476-76

Fig. 9.4. Poured-in Interior of Ridge-Pole Shelter at 15 psi. The main room was undamaged; the ridge pole had been moved only 3/4 in. away from GZ.

PHOTO
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In three places the underlying plastic over the ridge was broken; as a result, dry, sandy earth fell through the roof pole in these places, producing holes several inches across, open to the sky.

The seriousness of what would be the amount of blast-wind scouring by a 1-MT explosion (which at a given overpressure range would produce blast winds lasting 10 times as long as the 1-kiloton blast winds at DICE THROW) is indicated by the removal of all shielding earth from the GZ side of a 1/10-scale model of this Ridge-Pole Shelter, also tested at 15 psi (see Fig. 9.5).

PHOTO
6392-76

Fig. 9.5. Posttest Exterior of 1/10-Scale Ridge-Pole Shelter at 15 psi. Scouring by the blast winds had removed practically all the earth, plastic and twigs on the side facing GZ and over the two entries--indicative of blast-wind scouring of earth cover over a full-scale shelter by a megaton explosion.

The most suprising damage is shown by Figs. 9.6 and 9.7. Obviously, the dry earth "flowed" away from the middle of the shelter and toward the two ends of the shelter. Apparently the pressures on the ends of the shelter were decreased as compared to the pressures on the center, both by the lack of reflected overpressures at the ends and the lowering of pressures at the ends caused by Bernoulli effects where the velocities of the blast winds were increased as the winds passed around the ends of the obstructing shelter.

PHOTO
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Fig. 9.6. Collapsed Entrance Facing Away from GZ.
The blast winds had scoured away most of the covering
earth, and the earth had "flowed" away from the center
of the shelter, pushing the upper part of the entry in
a direction perpendicular to the radius from GZ.

PHOTO
6477-76

Fig. 9.7. Postblast View of the Entry of the
Ridge-Pole Shelter Facing GZ, at 15 psi. Note the
scattered potato-sack sandbags, that had been placed
to retain the earth over the entry. Earth "flow"
had pushed all but the base of the entry away from the
middle of the shelter, leaving none of the entry walls
perpendicular.

The three fireplace-size logs (see Fig. 9.7) in front of the entrance facing GZ, and also the two poles pictured resting on the side of the shelter, had been carried by the blast winds from where they had been stacked before the test at the 70-psi range, 315 ft from where they came to rest. Note the identifying spot of paint on the end of the log on the right.

The overpressure inside only reached 3 psi, due to the small size of the semicollapsed entryways, the relatively large volume of the main room, and the relatively short time (about 80 ms) that the overpressure outside remained about 3 psi.

9.3.2 - At 5.8 psi.--As anticipated, this Ridge-Pole Shelter was undamaged as regards its pole frame. Measurements showed the ridge pole to be unmoved. However, 6 to 12 in. of dry, sandy earth was removed from the ridge, partly due to blast-wind scouring and partly due to shock effects having broken 5 holes in the 4-mil polyethylene where the thin plastic covered the rough ends of the wall poles.* Some dry, sandy earth had fallen through these holes, but no part of the roof was wholly uncovered.

The overpressure measured inside was 2 psi.

9.4 Conclusions and Recommendations

9.4.1 - Due to the amount of dry shielding earth that would be removed by the blast winds that would be produced by the sizes of nuclear weapons that menace the United States, and also due to the damage that aboveground shelters built in wooded areas would suffer from blast-hurled trees, the practicality of Ridge-Pole Shelters for protection against both blast effects and fallout is severely limited. (If the earth is wet, however, blast-wind scouring by 1-kiloton blast winds at the 16-psi overpressure range removes a negligible thickness of sandy earth from a shelter with the same slope and orientation of roof.[1])

9.4.2 - Before covering this type shelter with thin plastic prepar-
atory to covering with earth, the ends of its poles should be covered
with cloth, rugs, or other stronger material.

10. DOOR-COVERED TRENCH SHELTERS AT 31 AND 15 PSI

10.1 Purpose

Most separate American homes have enough interior doors to roof a
trench shelter for the occupants and thus provide them with much better
protection against fallout radiation and fire than do the great majority
of homes. In a prior DNA blast test, a Door-Covered Trench Shelter was
essentially undamaged at 5 psi. Therefore, we tested this simple fallout
shelter at the predicted 30- and 15-psi overpressure ranges. The test
at 30 psi was carried out to learn whether or not earth arching would
prevent the collapse of the hollow-core interior doors roofing a trench
dug in almost rock-like earth--not to estimate the ultimate survivability
of persons exposed to 30-psi blast effects in a very small open shelter.

10.2 Construction

The Door-Covered Trench Shelters at 31 and 15 psi were of identical
construction, as shown by Fig. 10.1. However, a greater thickness of
earth was mounded over these shelters, about 2-1/2 ft, than shown in
this drawing. We had found that a hollow-core interior door can with-
stand being covered with earth many feet thick, since it yields under
loading and protective earth arching develops in earth mounded over it.

10.3 Test Results

10.3.1 - The shelter at 31 psi was a total failure. Earth arching
over the door did not prevent it from being broken in at this high
overpressure. Figure 10.2 shows the depression resulting from this
collapsed shelter, photographed eight days after the blast. Note the
large amount of sand that had been blown into this depression during these

Fig. 10.1. Door-Covered Trench Shelter Tested at 31 and 15 psi.
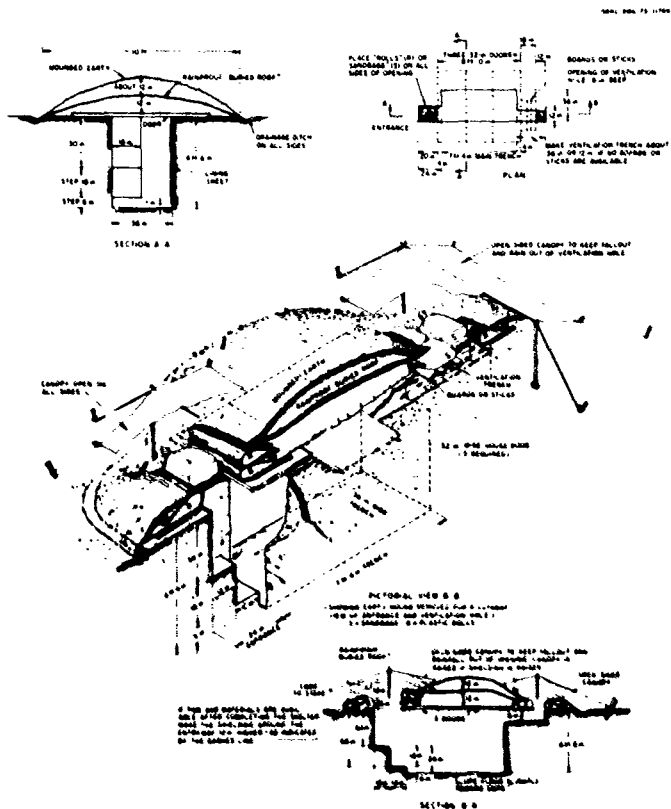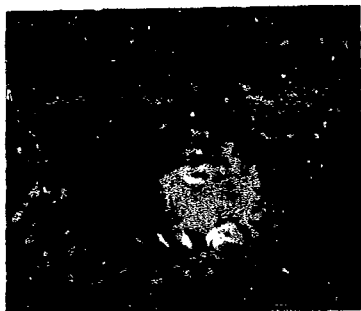
Fig. 10.2. Photo of Door-Covered Trench Shelter Taken
Eight Days After the Blast. The doors were smashed in. Note
the sand accumulation in the right side of the hole indicative
of the probability of dangerous amounts of fallout being blown into
entries used as ventilation openings in blast-devastated areas.

postblast eight days. In the desert outside the blast-devastated area,
the grass and desert shrubs prevented any consequential blowing of sand
and dust during these same eight days. Open entries serving as ventilation
openings had large amounts of sand blown into them, indicating a potential
fallout-entry problem in blast devastated areas.

10.3.2 - At 15 psi the roofing doors were cracked but not broken
in. (See Fig. 10.3.) However, much hard caliche was broken off the
walls. The overpressure measured was 5 psi, high enough to break some
occupants' eardrums.

10.4 Conclusions and Recommendations

10.4.1 - If subjected to the longer-duration overpressures and
greater amplitudes of ground motions caused by strategic weapons,

Fig. 10.3. Postblast Interior of Door-Covered Trench
Shelter at 15 psi. Large chunks of earth were knocked off
the walls. Between 16 and 24 days after the blast, the partly
broken doors broke completely.

Door-Covered Trench Shelters would afford obviously inadequate blast
protection at overpressure ranges considerably less than 15 psi.

10.4.2 - In blast-devastated areas, the problem of fallout par-
ticles being blown into shelters dependent for their air supply on
ground-level openires could be serious.

11. CHINESE "MAN" SHELTER AT 20 PSI

11.1 Purpose

In the first Chinese handbook[16] on nuclear defense that came into
our hands, we saw the shelter illustrated by Fig. 11.1. Previously, we
had never seen or conceived a blast shelter of this design or one built
of such thin poles. If such thin poles could safely be used, it would

reduce the labor of obtaining the poles for an expedient blast shelter--
one of the chief constraints on the practicality of such shelters.
Therefore, we decided to blast test this Chinese design.
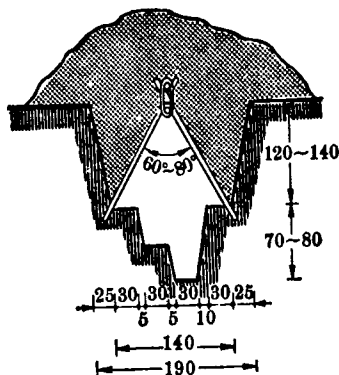
PHOTO
77-12599

图 3-107 人字形骨架避弹所

Fig. 11.1. Chinese "Man" Shelter Tested at 20 psi. This
shelter is called "Man" Shelter in a Chinese civil defense hand-
book[16] because a cross section of its frame resembles the Chinese
character "人" for "man."

11.2 Construction

The main room was 10 ft long. It was made in a trench with two
shelves, a bench and a 1-ft-wide foot-trench dug into the hard caliche.
The sloping wall poles were first cut 6 ft 6 in. long, but later had to
be reduced about 6 in. in length because their lower ends could not have
been dug into the rock-like earth without breaking off large chunks of
the two shelves on which the wall poles rested. The two small poles,

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one below and one above where the wall poles crossed at the top of the frame, were encircled tightly with a single strand of No. 9 wire between each adjacent wall pole.

The 10-ft-long main room (see Figs. 11.2 and 11.3) plus a 5-ft-long horizontal entryway required 28 poles on each side, averaging about 3 in. in diameter, including bark. The tops of these poles averaged about 2-1/2 in., excluding bark. The horizontal entryway was of the same design as the main room, except that its entire floor was at the same level as the shelves and the bottoms of the wall poles of the main room. It led to the vertical south-end entry that, for lack of a Chinese drawing, we designed and built using the triangular construction pictured in Figs. 11.2 and 11.4. The poles of the vertical entry averaged a little over 3 in. in diameter, including bark. Above the 30-in. x 30-in. opening at the outer end of the horizontal entryway, the inside of the vertical entry was an equilateral triangle 39 in. on a side--big enough for a big man. (See Figs. 11.2 and 11.4.) The five uppermost poles averaged 4 in. in diameter, and the top three were notched and nailed together so as to make a plane on which the blast door could be closed snugly.
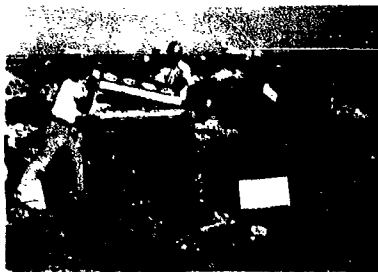
PHOTO
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Fig. 11.2. Completed Frame of Chinese "Man" Shelter Tested at 20 psi. In accord with the Chinese drawing, the poles of the main room averaged only about 3 in. in diameter. The triangular entries and triangular blast doors were of ORNL designs.
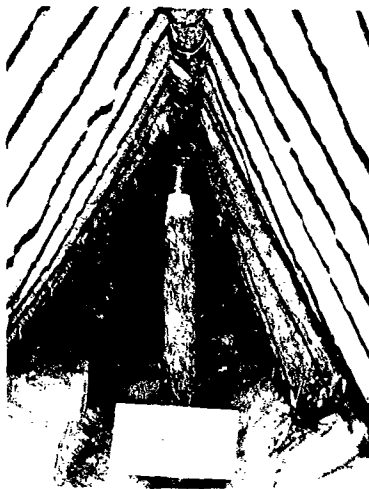
Fig. 11.3. Undamaged Interior, Showing Earth Bench on
One Side and Roof-Deflection Gauge on Post.

The blast door was very similar to the triangular blast doors on
the Log-Covered Trench Shelters described in Section 6, except that
the three frame poles of the door were smaller in diameter, and the
door had six open slots and six flap valves, as shown in Figs. 11.5
and 11.6. To prevent the door and the uppermost poles of the triangular
entry from being pulled up and blown away during the negative pressure
phase of an explosion, the uppermost poles were wired securely to poles
about 3 ft lower down the entry.

Fig. 11.7 shows the covering of the shelter frame, except for the
mounding of the shielding earth. Due to a construction error, the earth
was mounded 4 ft deep above the tops of the wall poles, rather than the
approximate 3 ft shown by the Chinese drawing.

PHOTO
2954-77

Fig. 11.4. The Lower Part of the Vertical Triangular
Entry is Pressed Horizontally Against Two Pairs of Vertical
Posts. Each pair is wired together. The two pairs are held
apart by two horizontal spacer poles toenailed in place to
frame the rectangular opening between the horizontal and
vertical parts of the entry. The pairs of vertical posts are
pressed against two horizontal poles (the uppermost is shown)
that in turn press against both the outermost two poles of the
horizontal part of the entry and the earth in two slots dug
in the sidewalls of the excavation.
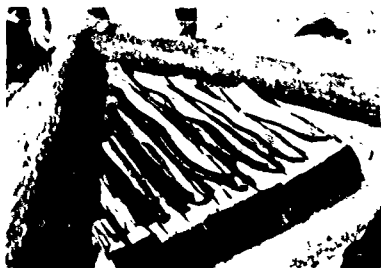
PHOTO
6467-76

Fig. 11.5. Posttest Undamaged Triangular Blast Door,
Made of Three 5-in.-Diameter Peeled Poles Covered with Seven
4-in.-Diameter Peeled Poles. Between these covering poles
were six 2-in.-wide ventilation slots, protected by six flap
valves made of strips cut from worn tire treads.

PHOTO
6469-76

Fig. 11.6. Undamaged Triangular Blast Door, Partly Open and Viewed Looking Up the Side of the Triangular Entry to Which the Door was Hinged.

PHOTO
6466-76

Fig. 11.7. Covering the Limb-Covered Pole Frame with Bedsheets. Salt cedar limbs had first been placed crosswise over the lightly constructed pole frame.

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The other (north) entry was ruggedly constructed of 6-in.-diameter
vertical poles, with interior triangular braces. Its blast door was
practically identical to the door on the ORNL-designed "Chinese" entry
to the south end of the shelter. A rectangular expedient shelter-
ventilating pump (a 20-in. x 24-in. KAP) was installed in a makeshift
frame placed in the horizontal crawlway leading to the north entry.

11.3 Test Results

Contrary to our predictions, this lightly constructed shelter,
tested closed, was undamaged by blast effects. The undamaged interior
is pictured in Figs. 11.3, 11.8 and 11.9.

PHOTO
0701-77

Fig. 11.8. Posttest View of Opening at Bottom of Triangular
Vertical Entry, Undamaged by 20-psi Blast Effects.

PHOTO
6470-76

Fig. 11.9. Postshot Condition of the Lightly Constructed
Triangular Vertical Entry. The hammer rests on a step-pole.
Earth arching prevented the yielding bedsheet outside from being
torn.

The triangular blast-protector logs around the doors, each 8 ft
long and 7 to 8 in. in diameter, were moved away from ground zero, so
that a log pressed against the blast door hinges of each door. See Fig.
11.5. Both doors, however, could be opened. The blast winds scoured
away about 8 to 10 in. of dry earth from around the six logs.

The blast valves on both doors obviously closed properly; a pres-
sure rise of only 1 psi was recorded in the center of the shelter. The
subsequently open valves permitted enough sand and dust to fall into the
entries to have constituted a health hazard if heavy fallout had been on
the ground outside. The ventilating pump and its flimsy frame were
damaged slightly, but required only about 10 minutes to repair before
postshot testing.

The undamaged shelter frame was moved only slightly. The top of
the roof was permanently depressed 1-5/8 in. and pushed 3/4 in. away
from GZ.

11.4 Conclusions and Recommendations

11.4.1 - The Chinese "Man" Shelter, if built with the ORNL-designed triangular vertical entry and expedient blast doors, is a good example of the blast protection attainable by properly building a lightly constructed shelter that yields under blast loading so as to attain effective earth arching in an adequately thick earth covering.

11.4.2 - We lack information concerning the magnitude and duration of the earth pressures produced by the blast on the wall poles--pressures that tend to collapse this A-frame structure. Therefore, we are unable even to hazard a prediction as to whether or not this closed shelter would survive the blast effects of a megaton weapon at the same 20 psi overpressure range, producing greater and much longer-lasting overpressures at depth, and ground waves of much greater amplitude.

11.4.3 - During a rapidly escalating crisis, in many wooded areas the most difficult poles to supply in adequate numbers at shelter-building sites would be the long, straight, stout poles required to make rectangular entries of blast shelters. Therefore, triangular blast entries made ot short, light poles and triangular expedient blast doors should be tested at higher overpressure and longer duration.

12. RUG-COVERED TRENCH SHELTERS AT 15 AND 5.8 PSI

12.1 Purpose

Tarp-Covered Trench Shelters had been undamaged by heavy static and moving loads, including a 6-ton backhoe driven over the earth

covering a shelter of this type roofed by a cotton tarp.[11] Since a
cotton tarp is not as strong as a piece of typical wall-to-wall car-
peting made largely of strong synthetic fibers, we anticipated this
shelter would withstand the blast at the 15 psi overpressure range, by
facilitating earth arching.

12.2 Construction

Fig. 12.1 shows the principal design elements of a Rug-Covered
Trench Shelter. The two models tested at DICE THROW had main-room
trenches 40 in. wide, 6 ft deep, and 11 ft long. The roofing rugs were
each 12 ft wide by 11 ft long. These rugs had a double laminated jute
backing over nylon--typical low-cost wall-to-wall carpeting. No diffi-
culties were experienced in covering the rugs with earth to a depth of
48 in. over the mid-line of the trench, nor in completing the 20-in.-
wide entrances at each end. See Figs. 12.2 and 12.3.

12.3 Test Results

12.3.1 - At 15 psi, the rug was torn lengthwise on one side from
end to end, and the mass of overlying earth fell into the trench. This
complete failure is shown clearly by Fig. 12.4.

12.3.2 - At 5 psi, the rug was not torn, but the ground shock
loosened it from the earth holding one of its edges in a side trench.
As a result, the whole untorn rug and the mass of earth above it fell
into the trench, to within about 18 in. of the trench floor. At this
point, earth arching and the strength of the rug stopped the downward
fall.

Occupants sitting in the trench would have been crushed.

12.4 Conclusions and Recommendations

12.4.1 - A Rug-Covered Trench Shelter definitely should not be
built in areas likely to be subjected to blast effects.

12.4.2 - A Rug-Covered (or Tarp-Covered) Trench Shelter should only
be built for fallout protection, in an area where the earth is very
stable, by persons lacking other materials with which to roof an expe-
dient trench shelter.
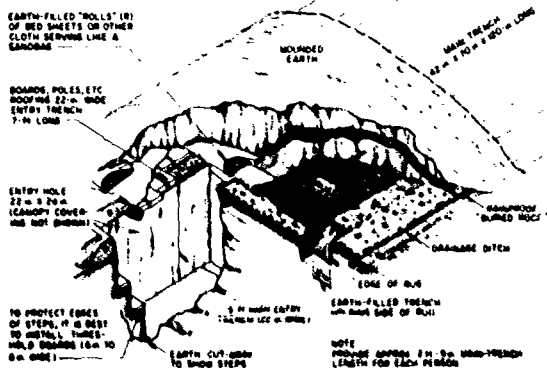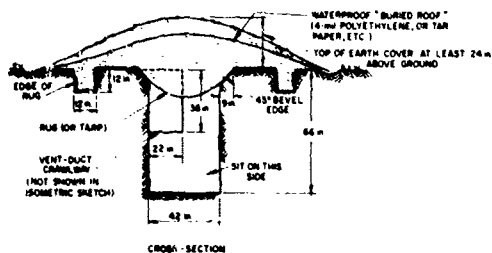
Fig. 12.1. Construction of Rug-Covered (or Tarp-Covered) Trench Shelter.

PHOTO
6490-76

Fig. 12.2. Tamping Earth over Edge of a Side-Trench of
Rug-Covered Trench Shelter at 15 psi.

PHOTO
6491-76

Fig. 12.3. Dumping Earth on Side of Rug before Mounding
Earth 4-ft Deep along Centerline. An earth-filled bedsheet
"roll" and a pillowcase "sandbag" retained earth at each entry,
pictured prior to completion.

58.

Fig. 12.4. Demolished Rug-Covered Trench Shelter at 15
psi. The edges of the rug were not pulled loose by blast
effects; it was torn lengthwise.

13. SCALE MODELS OF SHELTERS

13.1 Purpose and Construction

In order to save money and to compare the resistance to blast
effects of full-scale and reduced-scale shelters, the scale models
listed below were tested. All scale models were built of materials
as similar as practical to those of their full-scale counterparts, and
linear scaling of all dimensions was used in all cases.

13.2 Test Results

13.2.1 One-Half-Scale Rug-Covered Trench Shelters at 15 psi and
5 psi.--Both were undamaged by the blast effects, whereas their full-
scale counterparts failed at the same overpressure ranges. The canvas
used to roof the one-half-scale models was approximately one-half as
strong as the wall-to-wall carpeting used to roof the full-scale shelters.
For the one-half-scale models, a fabric only one-fourth as strong should
have been used, since the weight of earth supported by a 1-ft-wide

segment of the roofing fabric (measured along the edge of the trench) of
the one-half-scale model is one-fourth as great as the weight of earth
supported by a 1-ft-wide segment of the roofing fabric of the full-scale
shelter (1 x 1/2 x 1/2 vs 1 x 1 x 1). But even if we had selected
roofing fabric only one-fourth as strong for the one-half-scale model,
scaling would not have been satisfactory because the strength of the
earth of the unsupported walls would have remained the same in both
models, whereas the full-scale model to be equally strong would require
earth having twice the resistance to shearing and tensile stresses.

13.2.2 - Unshored Earth Walls of Trench Shelters.--In all cases, at
the same overpressure ranges the unsupported earth walls of small-scale
trench shelters and of small-scale open trenches were less damaged by
blast effects than were the corresponding walls of large-scale trench
shelters and of large-scale open trenches. This was due to the fact
that the volume of earth tending to be sheared off a trench wall by
gravity and ground-shock forces increases as the cube of the increase in
scale, whereas the area of the surface of the potential shearing-off of
this volume increases as the square of the increase in scale. As a
result of this difference, if we double all linear dimensions of a half-
scale trench, then in the case of the full-scale earth wall a unit-area
of the surface of potential shearing is subjected to twice the unit
stresses to which a corresponding unit-area of the half-scale earth wall
is subjected. Therefore, the full-scale trench wall fails first.

13.2.3 - One-Half-Scale Chinese "Man" Shelter at 31 psi.--This
closed shelter (Fig. 13.1) was a one-half-scale counterpart of the
Chinese "Man" Shelter tested at 20 psi, except that it had only one
entry and had only one blast-protector log, which was 10 in. in diameter
and secured by stakes.

Fig. 13.1. One-Half-Scale Chinese ⋀ ("Man") Shelter
Tested at 31 psi with Its Triangular Blast Door Closed.
Before being covered with earth mounded as high as the blast
door, the whole shelter was covered with 4-mil polyethylene.

The blast tore loose the blast-protector log. The blast winds,
theoretically peaking at about 670 mph, hurled this log 180 ft, where it
struck the side of the Ridge-Pole Shelter. About 10 in. of dry earth
were scoured from around its entry. The earth shelves on which the
lower ends of its side-poles rested were cracked, but not broken off.
About 2 in. of powdery caliche earth accumulated on the floor. The
height of the shelter roof was reduced only 7/8 in.

13.2.4 - One-Tenth-Scale Chinese ⋀ ("Man") Shelter at 31 psi.--
This 1/10-scale model consisted only of a main room, closed at both ends
with "poles," with its top at ground level. The frame was undamaged,
but had been pushed into the sandy earth 2 in., reducing the ceiling
height of the room from 4-1/4 in. to 2-1/4 in. (see Fig. 13.2). If a
full-scale shelter built in soft earth had its poles proportionally
pushed down into the earth by the 31-psi blast overpressure from a 1-MT
explosion, with its duration of overpressure ten times as long as from
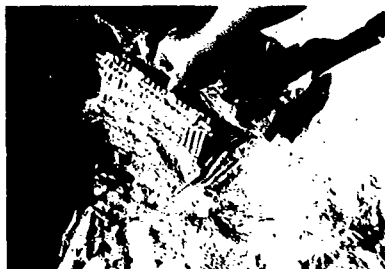
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Fig. 13.2. One-Tenth-Scale Room of Chinese "Man" Shelter
at 31 psi, photographed Posttest. The frame had remained
adequately covered, was undamaged, but had been pushed about
halfway into the ground.

a 1-kiloton explosion, the intact survival of the shelter frame would be
unimportant to occupants of this shelter.

13.2.5 - <u>One-Half-Scale Log-Covered Trench Shelter at 53 psi.</u>--This
closed shelter consisted solely of a two-level room and a horizontal
entry trench, counterparts of the adjacent full-scale Log-Covered Trench
Shelter. Both of these shelters were built to compare the effectiveness
of roofing a trench with poles laid on the surface of the ground as
illustrated in Russian civil defense handbooks, as compared to the
recommended Chinese procedure of placing the roofing poles on shelves
well below ground level. See Fig. 13.3.

The blast damage suffered by both parts of this closed shelter
indicated that occupants probably would have been injured, but was less
serious than the damage suffered by its full-scale counterparts tested
at 53 and 31 psi. In the Russian half, the upper parts of the earth
walls were broken off, and the unbroken roof poles came to rest sloping,
with a reduction of 3-3/4 in. in mid-ceiling height. In the Chinese
half, the roof poles remained horizontal, although they were lowered 2-
1/4 in. in the center.

Fig. 13.3. Construction of 1/2-Scale Log-Covered Trench
Shelter at 53 psi. The Chinese way of placing roofing poles
below ground level is shown in front; the Russian way, to the
rear.

13.2.6 - <u>One-Tenth-Scale Ridge-Pole Shelter at 15 psi.</u>--Unlike its
adjacent full-scale counterpart, its entryways were undamaged. However,
as shown by Fig. 9.5 of Section 9, the earth covering the side of its
frame facing GZ and the tops of its entryways were completely uncovered.

13.2.7 - <u>One-Tenth-Scale Small-Pole Shelters at 53, 106 and Approxi-
mately 180 psi.</u>--The shelter at 53 psi was undamaged, as was its full-
scale counterpart at 53 psi. The shelter at 106 psi failed; one of its
two vertical entries was wrecked, and lethal overpressures apparently
entered through its smashed entry (see Figs. 13.4 and 13.5). Neither
of the 6-in.-deep earth covers of these 1/10-scale shelters was seriously
wind-scoured. By contrast, their full-scale counterpart at 53 psi,
shielded by an earth mound with slopes of 36°, lost over 8 to 12 in. of
cover due to blast-wind scouring. However, the shielding earth over
the 1/10-scale models was mounded with slopes of only about 10°, and the

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Fig. 13.4. One-Tenth-Scale Small-Pole Shelter, Pre-
test at 106 psi. Earth was mounded over this shelter at
slopes of about 10° to minimize blast-wind scouring. Only
the plywood blast doors are visible.

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Fig. 13.5. One-Tenth-Scale Small-Pole Shelter, Posttest
at 106 psi, Shown after Being Carefully Uncovered.

wind velocities a few inches above the quite rough ground were not as
high as those striking the 5-ft-high mound over the full-scale shelter.

At the approximately 180-psi overpressure range, a 1/10-scale model of only the main room of a Small-Pole Shelter, tested closed and covered with 6 in. of unmounded sandy soil, failed. The wall poles were pushed down about one-third their heights, and the lower cross-bracing "ladder" broke, with poles left sticking upward into the living space.

13.3 Conclusions

13.3.1 - The successful testing of a reduced-scale shelter does not justify an assumption that its full-scale counterpart will survive as well in the same blast environment, especially under the dynamic loadings produced by large explosions.

13.3.2 - When the critical stresses in the test structures (including stresses in earth banks subject to failure by shear) are induced by gravity and/or the acceleration or deceleration of masses, these stresses in the model are reduced by the scale factor.

14. BLAST-HURLED DEBRIS

14.1 Purpose

Blast tests have very rarely involved simulating the conditions of urban, suburban, or wooded areas as regards the damage likely to be caused by blast-hurled debris. Structures that could easily be damaged by heavy projectiles have frequently survived shock waves and blast winds because no materials to simulate houses and trees were placed between them and GZ. See Ref. 3 for examples. Small expedient shelters, especially aboveground types and shelters with small, steeply-sloped earth coverings, could be damaged or destroyed by blast-hurled heavy projectiles such as tree trunks or the parts of houses.

Therefore, to get at least a feeling for the magnitude of this neglected problem, we secured permission to expose to the blast some fireplace-sized logs, left-over lumber, a 14-ft-high complete tree "planted" securely in the hard caliche, and three 16-ft 2 x 4's also "planted" securely. Most of the logs were stacked in a woodpile at the approximately 70-psi range, with the logs pointing toward GZ. Six logs averaging 8 in. in diameter were placed on top of the 5-ft-high mound of earth over shelters at 53 psi. The logs and boards were marked with paint of different colors, for posttest identification.

14.2 Test Results

The shock wave and blast winds hurled this debris farther than the standard blast wind velocities and theoretical calculations would lead one to believe. Most of the fireplace-sized logs came to rest 240 to 360 ft from their starting positions and seven were airborne between 360 and 640 ft. The farthest airborne, a 5-in.-diameter, 18-in.-long stick, came to rest 640 ft from the woodpile. Fourteen logs struck the 5-ft-high mound over the Log-Covered Trench Shelter at 53 psi and were imbedded in the soft earth, as pictured in Fig. 14.1. Of the 73 pieces of blast-hurled debris that were found, 33 pieces were hurled between 240 and 360 ft and came to rest between approximately the 19- and 13-psi overpressure ranges.

The 14-ft-high tamarisk (salt cedar) tree, cut and "planted" two days before and still in full leaf, was broken off at the ground. Apparently, it was broken into very small pieces, and the pieces carried far away, since we were unable to find any part of this tree. The three vertical 2 x 4's were each broken into two or more pieces, some as short as 2 ft long.

Two of the small logs were hurled end-on into the earth bank over the shelters at 53 psi and punched into the bank. See Fig. 14.2. Most of the logs apparently bounced upward on hitting this bank sloping toward

66.

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Fig. 14.1. Some of the Fireplace-Sized Logs Hurled from a
Woodpile and Imbedded in the 5-ft-High Mound at 53 psi. Apparently,
the blast winds of the negative phase had uncovered the two small
logs in the foreground and moved them toward GZ.

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Fig. 14.2. Posttest Condition of the Side Facing GZ of the
5-ft-High Mound of Earth over Log-Covered Trench Shelter at 53 psi.
The log sticking out of the mound had been hurled by the blast winds.
The canvas had been covered with about 4 in. of earth, in a marginally
successful attempt to reduce blast-wind scouring.

GZ at about 36° and were swept higher upward by the turbulent blast
winds. Some came to rest when they struck shelter mounds farther from
GZ, as is shown in Fig. 9.7 of Section 9.

14.3 Conclusions and Recommendations

14.3.1 - Blast-hurled debris would constitute a serious hazard to
most expedient shelters built in areas of the types where most Americans
live or would evacuate into during a nuclear crisis, if these areas were
subjected to severe blast effects.

14.3.2 - For reasons explained in Section 17, "Limitations of the
DICE THROW Tests," it is extremely difficult to estimate from this
evidence (based on a 1-kiloton air blast) the much greater hazards from
blast-hurled debris likely to result at the same overpressure ranges
from strategic nuclear weapons.

15. EXPEDIENT WATER STORAGE

15.1 Purpose

For a shelter to be occupied for weeks in an area of severe fallout
hazards, adequate drinking water must be available close at hand. The
survivors in areas likely to be subjected to both blast effects and
heavy fallout should not depend on normal sources of drinking water or
on water stored in containers likely to leak as a result of blast
effects. Therefore, we conducted the first blast tests of simple,
inexpensive expedient means for storing many gallons of water per
shelter occupant.

15.2 Construction and Test Results

15.2.1 - Water Stored in Plastic Bags Lining Cylindrical Pits in
the Earth.[11]--As anticipated, lined cylindrical pits proved to be the
most blast-resistant way to store water outside of blast shelters. See
Fig. 15.1. Ordinary 30-gal polyethylene trash bags were used for water-
proof liners. One bag was placed inside of another, since a very small

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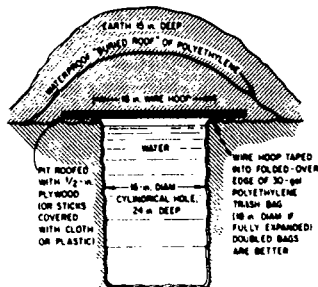
Fig. 15.1. Vertical Section of Cylindrical Water-Storage
Pit Lined with Waterproof Plastic Bag, or Two Bags.

fraction of polyethylene bags that are not made for water storage have
pinhole leaks. Each cylindrical pit was dug so as to have a diameter
about 2 in. smaller than the diameter of its waterproof liner-bag, when
its liner-bag was inflated.

The best way to keep the upper edges of the pit-lining bag from
slipping into a pit is illustrated by Fig. 15.2: Make a circular wire
hoop the size of the mouth of the bag, and tape it into the mouth. This
method was used in the water-storage pits at the 20-psi and 6.7-psi
overpressure ranges. At the 53-psi range, the upper edges of doubled
bags were satisfactorily held in place merely by sticking six 4-in.
nails through the turned-under edges of the bags and into the very firm
earth.

Before the test, the lined pits, each approximately 2 ft deep,
were filled almost full, and then roofed and covered as illustrated by

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Fig. 15.2. Cylindrical Water-Storage Pit Lined with
Two Polyethylene Trash Bags. After exposure to blast effects
at 20 psi, this pit was undamaged and still full of water.

Fig. 15.1. Each lined pit contained about 20 gal of water. The earth
cover was sufficiently thick to result in very effective earth arching
under the blast loadings; both plywood pit-roofs were cracked but not
broken. None of the three storage pits developed leaks. Even at the
53-psi range, the blast effects resulted in no caving of the pit wall.

The storage pit at 53 psi, that after the blast was left partly
open to the dry desert winds, showed only 4% loss of water after 8 days.
At the 20-psi range, after 24 days during which the pit was left com-
pletely open to the dry desert winds, it was about 70% full, and at
6.7 psi, the covered pit had only lost about 4% of its water after 24
days.

15.2.2 - Water Stored in One or Two Plastic Bags Used to Line a
Smaller Fabric Bag or an Ordinary Pillowcase.--This method can be used
to transport and store quite large volumes of water.[11] Two burlap
potato bags, each lined with two 20-gal polyethylene trash bags, were
each filled with about 10 gal of water.

One of these expedient containers was tested inside the Small-Pole Shelter at the 53-psi overpressure range. Its mouth was tied shut with a 1/4-in. cord, one end of which was then tied to a nail driven into a wall pole of the shelter, about a foot above the top of the water bag. This cord kept the mouths of the burlap bag and its double lining bags above the level of the water inside.

This water storage was unaffected by the quite severe ground shock inside the closed shelter.

Inside the open Russian Pole-Covered Trench Shelter at 20 psi, an identical water storage container was undamaged by the shock wave and blast winds that entered through the open stairway.

15.2.3 - <u>Water Stored in Plastic-Lined Trenches.</u>--Figure 15.3 is a postshot photo showing a lined water-storage trench at 6.7 psi. This

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Fig. 15.3. Postshot View of Plastic-Lined Water-
Storage Pit at the 6.7-psi Overpressure Range.

trench was dug 8 ft long, 27 in. wide, and 30 in. deep, and had been lined with a 10-ft-wide sheet of 4-mil polyethylene, with its edges secured in small, earth-filled ditches. About 200 gal had filled it to within about 6 in. of full. The pit had then been covered with the pictured 3/4-in. plywood sheets. Earth had next been mounded about

30 in. deep over the plywood, incorporating a waterproof "buried roc" to keep out fallout-contaminated rain water. The resulting cross-sectional profile was similar to that shown in Fig. 15.1.

Ground shock resulted in some earth caving off the edges of the long sides of the trench, but no puncturing of the plastic lining resulted. Eight days after the blast, this side-wall caving had increased, but the trough still held a calculated 190 gal of water.

At the 20-psi range, a similar lined water-storage pit was badly damaged by side-wall caving, although earth-arching saved its roof. Before it could be examined after the blast, almost all of its approximately 200 gal of water had leaked out.

15.3 Conclusions

15.3.1 - If blast is expected in a shelter area, plastic-lined cylindrical pits, filled almost full and protected from blast and contamination as illustrated in Fig. 15.1, would usually be the most practical method of expedient water storage.

15.3.2 - Inside blast shelters, sufficient water for several days shoul- be stored in fabric bags lined with larger plastic bags.

16.   EXPEDIENT VENTILATION OF BLAST SHELTERS

16.1 Purpose

Expedient shelters that afford good protection against both blast and fallout have small entries, usually vertical. Such entries result in inadequate natural ventilation when a wind is not blowing. In hot weather, especially if it is humid, even with a breeze outside, a fully-occupied shelter can become dangerously or lethally hot and humid. Furthermore, we recognized the fact that air-intake and air-exhaust openings at ground level, if used for air supply in a blast-devastated area contaminated with heavy fallout, might have dangerous amounts of fallout blown into them. See Fig. 10.1 in Section 10.

The problem of pumping sufficient air through expedient blast valves of the types described earlier in this report needed investigation.

16.2 Observations, Construction and Test Results

Intermittently during the three weeks following the main event, we observed the amount of sand and dust that was added to the amount that came through the poorly-positioned blast valves in blast doors. Although in an area of very heavy fallout the amount that entered through these valves could prove serious, much more fell into the open entries of the shelters not partially protected by blast doors and the blast-protector logs around them.

The Small-Pole Shelter at 53 psi, which had solid plywood doors that had to be left partly open to secure adequate ventilation, presented a special problem. In an attempt to keep sand particles out, we built an improvised one-foot-high "wall" of sticks covered with polyethylene around the vertical entry, inside the blast-protector logs, and over the whole entry we erected an expedient tent. These measures reduced by about 60% the amount of sand subsequently blown into the shelter. However, if the area had been covered with heavy fallout, it would have been clearly impractical to work outside to install this "wall" and tent, even if all parts of the "wall" and tent had been carefully made to fit the opening before the blast, and stored inside the shelter for postattack use.

Postblast ventilation tests, using expedient KAPs and making air velocity measurements with a Hastings anemometer, yielded the following results:

(a) In the Log-Covered Trench Shelter at 53 psi, using a 20-in.-wide x 36-in.-high KAP (see Fig. 16.1), 412 cubic feet per minute (cfm) were pumped through the shelter when its blast doors were open; 177 cfm were pumped through the shelter with its two blast doors closed, with the air flowing through the blast valves. In each case a deduction was
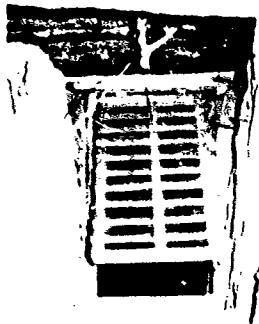
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Fig. 16.1. Expedient Shelter-Ventilating Pump (a 20-in. x
36-in. KAP) in an entry of the Log-Covered Trench Shelter at 53
psi. Tested preblast, it pumped 177 cfm through the valves of
the two closed blast doors and 412 cfm with the doors open. This
entry was demolished by blast effects.

made for the small measured volume of air that moved through the shelter
during times when the wind was blowing outside. Each door had blast
valves with openings totaling about 80 sq. in. in cross-sectional area.

(b) In the Chinese "Man" Shelter at 20 psi, using a 20-in.-wide x
24-in.-high KAP (see Fig. 16.2), with the two triangular blast doors
open, 350 cfm were pumped through the shelter with the blast doors
open; 240 cfm were pumped through the blast valves with the blast doors
closed. Each door had valves with openings totaling about 115 sq. in.
A gusty wind outside made these measurements less reliable, probably on
the high side.
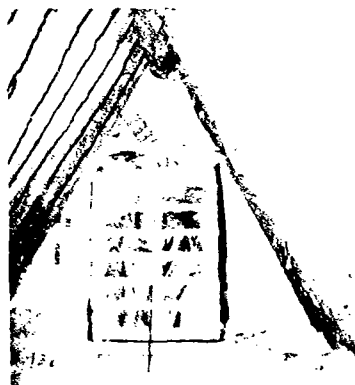
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Fig. 16.2. Expedient KAP (20 in. x 24 in.) Tested in the
Chinese "Man" Shelter at 20 psi, After the Blast.

(c) In the Small-Pole Shelter at 53 psi, using a 29-in.-wide x 36-
in.-high KAP when there was no wind outside, 861 cfm were pumped through
the shelter while the two solid blast doors were each open about 1 ft,
providing two openings each about 5 sq. ft in cross-sectional area. The
fallout-protective "walls" and expedient tent were around and over the
air-intake entry during this test. (A similar test conducted before the
blast, but with the doors completely open, resulted in a measured airflow
of 876 cfm.) See. Fig. 4.6.

16.3 Conclusions and Recommendations

16.3.1 - Blast valves in blast doors are impractical. If valves of
the type tested are mounted in separate vertical ventilation shafts, as
was done in the ORNL tests in DNA's MIXED COMPANY main event (see Ref.
1), the entry of fallout particles appears likely to be reduced

below dangerous levels. Ways to build expedient ventilation shafts that do not require heavy lumber should be developed and tested.

16.3.2 - Except in extremely hot and humid weather, an air supply of about 10 cfm per shelter occupant is enough to maintain tolerable conditions during continuous occupancy for several days. Therefore, even a KAP as small as 20 in. x 24 in. would usually prove adequate for a 15-man shelter protected by blast valves having total openings as large as those of the blast valves tested in DICE THROW (around 100 sq. in.) but installed in separate air-intake and air-exhaust ventilation shafts.

16.3.3 - Simple, expedient equipment to enable shelter occupants to raise ventilation air-intake and air-exhaust openings above ground level after the blast, and at the same time to quickly seal off the rest of the entries, should be developed and tested.

16.3.4 - For use in prefabricated blast shelters or in blast shelters that may be built in normal times or during slowly worsening crises, ventilation pipes that are installed with their upper ends safely below the earth until after the blast, and that can be raised by a jack above ground level after the blast, should be developed and blast tested. (Since DICE THROW, we have designed and built a prototype of such an extendable ventilation pipe, and also a manually operated, homemade suction pump capable of pumping around 60 cfm through a 3- or 4-in. pipe.)

17. LIMITATIONS OF THESE DICE THROW TESTS

Caution should be used in extrapolating from the results of these DICE THROW tests to estimate the survivability of expedient shelters-- especially those built in typical urban, suburban or wooded areas--if subjected to the blast effects of a large nuclear weapon, for the follow-ing reasons:

a. This blast was small, with airblast effects roughly equivalent
to a 1-kiloton nuclear explosion. At locations receiving the same peak
overpressures from a multimegaton surface burst, much more severe blast
effects would result, because:

(1) The duration of the overpressures and the dynamic
overpressures would be much longer (20 times as long from an
8-MT explosion),[3] and the energy transmitted to structures on
and below the surface could be many times greater. At the same
maximum overpressure ranges, the resulting destructive effects
from an 8-MT explosion on deeply buried parts of shelters and
the unshored earth walls of shelters would be greater. Also
the earth-flow phenomena observed (to a relatively minor extent
in some of these DICE THROW tests) would certainly increase in
some areas.

(2) The damages due to ground shock would be more extensive
due to the greater amplitude of the ground wave and (in the case
of an 8-MT burst) to the 20-fold greater distances from ground zero
to a given overpressure range. These greater distances usually
would permit the ground shock to arrive at the ranges of interest
up to hundreds of ms in advance of the air shock wave;
this difference between arrival times would cause the shelter roof
supports to be accelerated upward before any downward forces
from the airborne shock wave could cause downward movement of
the earth covering a shelter. The vertical amplitude of such
initial ground-shock (ground-wave) effects can be several inches,
and the inertial mass response of the earth covering a shelter
roof would thus cause the roof members to be bowed downward, to
an extent not observable in high-explosive tests of similar
shelters at similar overpressure ranges.

(3) Earth scouring of above-grade mounds by the blast winds
(that from an 8-MT explosion would blow for about 20 times as long
as from this "1-kiloton" DICE THROW shot) could be much greater

depending on the contour of the mound. Especially if the shielding earth were dry, such long-duration blast winds could blow away much of the shielding earth mounded above ground level over a shelter, possibly reducing its usefulness as a fallout shelter.

(4) Blast-hurled heavy projectiles--including the trunks of large trees and parts of houses and other structures--can be accelerated by a 1-kiloton explosion to velocities only a small fraction of those to which the same objects, if at the same over-pressure range, would be propelled by a multimegaton explosion. Persons estimating blast damage should remember that an object's kinetic energy varies as the square of its velocity. Furthermore, a hurled object 10 times as large as a small object having the same velocity, density, and relative proportions, and impacting in the same relative position on a fixed object, delivers 10 times the amount of energy per square inch of impact area. Therefore, the impact damage to be expected from large objects accelerated by a multimegaton blast cannot be accurately estimated from the results of experiments like those at DICE THROW nor from the damage caused by blast-displaced heavy objects at Hiroshima and Nagasaki.

b. Blast tests of scale models of shelters can give misleading results regarding the survivability of full-scale shelters subjected to the same blast effects. In the DICE THROW tests, all of the reduced-scale models of shelters withstood blast effects better than the corre-sponding full-scale shelters. For example, the half-scale Rug-Covered Trench Shelters tested at the 15- and 5.8-psi range both were undamaged, whereas the full-scale models both failed at the same overpressure ranges.

c. The earth was extremely stable in the DICE THROW test area. At almost all of the ORNL DICE THROW shelter sites, at depths of only a few inches the sandy desert soil changes to very stable caliche. At the 53- and 31-psi ranges, the hardness of this soil largely composed of sand grains cemented together with gypsum approached that of a very soft

limestone rock. Thus if shelters were built in typical habitated areas--that have much less stable soils--and were subjected to blast effects similar in magnitude to those at DICE THROW, the collapse of the unshored walls of trench shelters, the pressures exerted on deeply buried parts of shelters, and the earth-flow effects would all have been more pronounced and damaging.

18. PRINCIPAL CONCLUSIONS AND RECOMMENDATIONS

18.1

Expedient shelters of the types tested--especially if the ones with shored walls are equipped with blast doors--would afford better protection against the blast and fire effects of nuclear weapons and much better fallout protection than do all but a small fraction of existing buildings.

18.2

Ground-shock effects--not overpressure effects--would cause the failure of most expedient shelters with sufficient earth covering to assure effective earth arching. (In order to assure effective earth arching, the earth covering should be at least one-half as thick as the free span of the shelter roof. Also the roof and/or the whole structure must yield when loaded--thus causing the resultant earth arching around the structure to bear most of the load.)

18.3

Even in very stable ground, unshored trench shelters would be unsafe if subjected to the blast effects of large nuclear explosions at overpressure ranges of more than about 7 psi.

18.4

When roof cover is adequate to assure earth arching, flexible poles considerably smaller in diameter than those used to roof the ORNL unshored shelters should prove adequately strong.

18.5

Shelters likely to be subjected to blast effects should be built, whenever practical, with their roofs far enough below ground so that the tops of their entrances are no more than a foot above ground level. This positioning would greatly reduce blast damage and the removal of shielding earth by blast winds.

18.6

Expedient blast doors--especially doors made of poles and of triangular design--can be readily built strong enough to withstand as severe blast effects as the strongest expedient shelters tested to date.

18.7

Since the ground shock and earth flow effects from large nuclear weapons were not well simulated by the DICE THROW blast, expedient shelters and their life-support equipment should be tested under conditions producing much more severe blast effects on below-ground structures. The Air Force Weapons Laboratory's 125-kiloton Dynamic Air Blast Simulation (DABS) test planned for April 1978 should provide a realistic blast environment.

18.8

Means for assuring adequate and safe ventilation-cooling of shelters after they have been subjected to severe blast effects is the most neglected essential component of shelter design. Simple air-intake and air-exhaust openings, that shelter occupants could raise above ground level _after_ the blast and that would enable them to pump sufficient air through their shelter while excluding dangerous amounts of fallout, should be developed and blast tested.

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28. EXPERIMENTAL STUDY OF AIRCRAFT STRUCTURAL RESPONSE TO BLAST - DICE THROW PROJECT NO. 118

by

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Naval Weapons Evaluation Facility

ABSTRACT

In a continuing effort to improve the survivability of aircraft to nuclear overpressure effects, the Naval Weapons Evaluation Facility (NWEF) fielded three parked A-4C aircraft in the Defense Agency (DNA) sponsored DICE THROW high-explosive event. This test shot, simulating a 4.2 TJ (1 KT) nuclear surface burst, was detonated 6 October 1976. The primary objectives of the NWEF experiment were to provide overpressure response data on parked aircraft structures and compare this test data with analytic predictions. These objectives were met through tape recorded pressure load and structural response data on a variety of structural elements.

Complete verification of the predictive techniques is necessarily a statistical problem; however, within the scope of this present effort, the specified accuracy of the analytic procedures have been verified.

This report points out some significant problems in the areas of computer modeling of a true-to-life structure and in the use of classical reflected blast loading for complex non-idealized shapes.

The three increasingly sophisticated analytic methods from the DNA-2048 Handbook, accordingly, give more accurate predictions; however, each is more time consuming and costly to implement. One method is not recommended over another because the analyst must make the trade-off of accuracy versus cost.

The test is assessed as an excellent means for observing various modes of structural response to overpressure. Considerable data has been compiled on (1) pressure loading, (2) structural response of panels, longerons and frames, (3) rigid body motion and tiedown restraining forces and (4) synergistic effects of overpressure and gust. This data is available , with DNA approval, for more detailed correlations and predictions of aircraft vulnerability.

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TABLES

ABBREVIATIONS AND ACRONYMS

A/C	aircraft
ANFO	ammonium nitrate fuel oil (mixture)
BUNO	bureau number
C	Celcius (degrees)
dc	direct current
DEPROB	Dynamic Elastic Plastic Response of Beams (code)
DEPROP	Dynamic Elastic Plastic Response of Panels (code)
DNA	Defense Nuclear Agency
E	Young's Modulus of Elasticity
FM	frequency modulation
ft	foot
ft-lbs	foot-pound force
Fty, Fy	tensile yield stress
g	grams
G	shear modulus
GPa	gigapascal (10^{+9} Pa)
GZ	ground zero
Hz	hertz
in.	inch
J	joule
kHz	kilohertz (10^{+3} Hz)
kJ	kilojoules (10^{+3} J)
kN	kilonewton (10^{+3} N)

kPa	kilopascal (10^{+3} Pa)
KT	kiloton (10^{+3} metric tons (2208 lb))
lb	pound
lbf	pound-force
m	meter
mm	millimeter (10^{-3} m)
Mg	megagram (10^{+6} g)
MPa	megapascal (10^{+6} Pa)
ms	millisecond (10^{-3} s)
mV	millivolt (10^{-3} V)
N	newton
NOVA	Nuclear Overpressure Vulnerability Analysis (code)
NSWC/WOL	Naval Surface Weapons Center, White Oak Laboratory
NWEF	Naval Weapons Evaluation Facility
Pa	pascal
psi	pounds per square inch
s	second
t	time
TJ	terajoule (10^{+12} J)
V	volt
WSMR	White Sands Missile Range
μin/in	microstrains (microinch per inch)
ϵ	strain
ν	Poisson's ratio

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INTRODUCTION

The determination of aerospace structural response and vulnerability to nuclear blast effects under sure-safe and sure-kill criteria are the subjects of numerous analytic methods which have been developed for that purpose.

More recently, considerable effort has been spent on experimental studies which involved structural elements subjected to blast effects (Ref. 1). These studies, conducted on structural elements which were fabricated and tested under rigidly controlled conditions, provided useful results for correlating experimental and analytic data. This offered a measure of success in efforts to verify the validity of the analytic methods. The level of success varied, of course, with the accuracy of structural modeling, the blast representation, and the general sophistication of the analytic methods employed.

Despite these efforts, the pertinent questions which remain largely unanswered are:

1. How accurately does an idealized test structure represent a "true-to-life" airframe of an aerospace vehicle?

2. How does the response of a structural element incorporated within the aircraft structure compare to the response of an isolated, representative structural element when both are subjected to the same blast environment? - i.e., can an entire structure be modeled by a few representative elements?

3. Do current analytic techniques for airblast vulnerability accurately predict structural damage?

The Defense Nuclear Agency (DNA) sponsored DICE THROW program, which culminated in the detonation of a 564-metric-ton ammonium-nitrate-and-fuel-oil (ANFO) charge (4.2 TJ or 1 KT nuclear blast equivalent) on 6 October 1976, offered a unique opportunity for an experimental study of a complete aircraft structural response to a blast environment. Within the framework of that program, under DNA contract the Naval Weapons Evaluation Facility (NWEF) fielded three A-4C aircraft with the primary objective to obtain experimental data

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of the structural response to overpressure for correlation
with the analytic data derived from computer methods pre-
sented in the DNA-2048H-1 handbook (Ref. 2).

The description of the test program and the pertinent
analyses and subsequent correlation of the experimental and
analytic data are the subjects of this report.

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OBJECTIVES

The primary objective of the DICE THROW Project No. 118 study was to experimentally verify the overpressure analysis techniques contained in the "Handbook for Analysis of Nuclear Weapon Effects on Aircraft," DNA-2048H-1 (Ref. 2). Subordinated to this broadly defined overall objective, several intermediate objectives were formulated as follows:

1. To collect experimental data from a suitably instrumented A-4C aircraft subjected to a blast environment generated by the detonation of the ANFO charge which closely simulated a 4.2-TJ (1 KT) nuclear detonation.

2. To conduct appropriate analyses of the aircraft structural response to overpressure using the three analytic methods described in the handbook.

3. To correlate of the experimental and analytic data in order to determine the degree of agreement between the predicted and the actual response of the aircraft structure to overpressure, i.e., the actual verification procedures.

In keeping with the scope of these methods, the instrumentation system for the experimental data collection was designed to provide pressure, strain, and deflection data which could be directly correlated with the analytic data.

An experimental program of this magnitude presented unique opportunities, and two secondary objectives were postulated:

1. To determine the actual aircraft motion during the positive phase of the blast.

2. To assess the resulting impulse absorbed by the aircraft.

These secondary objectives were realized through analyses of the motion picture records in conjunction with the data obtained from the calibrated load cells incorporated in the aircraft tiedown system.

In addition to these clearly defined objectives, the test was seen as a means of obtaining specimens for future studies on the effects of changes in the aerodynamic characteristic of the aircraft structure. It was envisioned that the aircraft components which sustained damage approaching a catastrophic failure, for example, extensive buckling of a skin panel, could be used in a test designed to determine the aerodynamic drag increase related to the sustained permanent deformation.

An extension of the data correlation objective to the relevant data which were obtained from past nuclear tests was also included among the objectives. It promised to provide an informative comparison survey of the available experimental data.

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TEST LAYOUT

The definition of the test layout, i.e., positioning the aircraft with respect to ground zero (GZ), presented a singular problem arising from the lack of reliable data on the actual vulnerability of aircraft structures to overpressure. This was compounded by the fact that, in line with the objectives of this project, the desired structural response called for an overpressure level sufficient to produce permanent damage to the structural elements selected for instrumentation. Even more severe local damage, short of catastrophic failure, was considered necessary to provide adequate means for assessing the drag increase resulting from changes in the aerodynamic characteristics of the structure as a whole.

Earlier participation by NWEF in the PRE-DICE THROW II Event, described in Appendices A and B, consisted of a test on an instrumented F-4 wing trailing edge flap and was envisioned as the means for

1.   An initial endeavor to correlate analytic and experimental overpressure induced structural response.

2.   Shakedown and debugging of test instrumentation.

The NWEF test was only partly successful, even though it produced satisfactory experimental data. However, inasmuch as it failed to verify the presence of high strain levels predicted by the analysis, the problem of selecting locations for the aircraft for the main event remained unresolved. A certain amount of intuitive judgement based on related test programs had to be applied to arrive at suitable placements for aircraft within the blast field.

The final decision was predicated on information available from an experimental program conducted at the Naval Surface Weapon Center, White Oak Laboratory (NSWC/WOL), Maryland, which involved flat panels subjected to blast in a shock tube (Ref. 3). Also considered was the widely accepted notion that, according to the sure-safe criterion, skin panels should be regarded as the structural elements more vulnerable to the overpressure effects than the substructure elements such as stringers, longerons, or frames. With that in mind, 41.4 and 62.1 kPa (6.0 and 9.0 psi) were selected as the free-field overpressure levels which should produce the desired severity of damage in the fielded aircraft.

The general view of the aircraft emplacement is presented in Fig. 1. GZ marks the location where the 564-metric-ton ANFO explosive charge was assembled. The charge formed a cylinder of 4.27-m (14.0 ft) radius with a semispherical dome and a total combined height of 10.94 m (35.9 ft) and was designed to simulate the blast field from a 4.2-TJ (1 KT) nuclear ground burst.

The fielded aircraft were oriented side-on to the oncoming shock front. Two were placed 375 m (1230 ft) and one at 311 m (1020 ft) from GZ. These locations corresponded to the estimated ranges for free-field peak overpressures of 41.4 and 62.1 kPa (6 and 9 psi), respectively (Ref. 4).

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FIGURE 1. Scenario Layout

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AIRCRAFT CONFIGURATION

The A-4C aircraft (Fig. 2) assigned for the DICE THROW project were bureau numbers (BUNO) 149558, 145062, and 145074. These aircraft, each with approximately 4000 in-service flying hours and designated for retirement from the Navy inventory, were flown to NWEF. Upon their arrival, engines, pilot seats, and all avionics components were removed in compliance with the salvage requirements. A general inspection of the airframes was subsequently performed and aircraft BUNO 149558 was selected for the test instrumentation.

After the instrumentation was installed, the aircraft were transported by truck to the White Sands Missile Range (WSMR) test site and placed on 4.9 x 6.7 m (16 x 22 ft) pads (Fig. 3).

It was intended that aircraft BUNO 149558 and 145062, both 375 m (1230 ft) from the GZ, would demonstrate repeatability of damage for similar structures under practically identical blast environment. It was expected that the third aircraft BUNO 145074, 311 m (1020 ft) from the GZ, would sustain considerably more damage, bordering on catastrophic failure.

The desired test configuration for the aircraft was achieved through a series of operations which included:

    1.   Pressurizing the landing gear struts with nitrogen gas to simulate the parked aircraft attitude.

    2.   Setting and locking all flying control surfaces in a neutral position.

    3.   Filling internal fuel tanks with water.

    4.   Installing tiedown system.

An external fuel tank filled with 1136 liters (300 gallons) of water was installed at the centerline store station of aircraft BUNO 145074, i.e., the aircraft closest to the GZ. This extra mass was added to observe the effect of blast on external stores.

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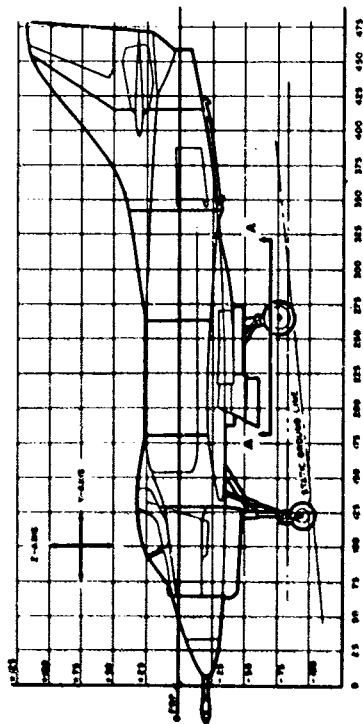
NOTE: Numbers denote inches.

FIGURE 2. A-4C Aircraft Configuration - Side View

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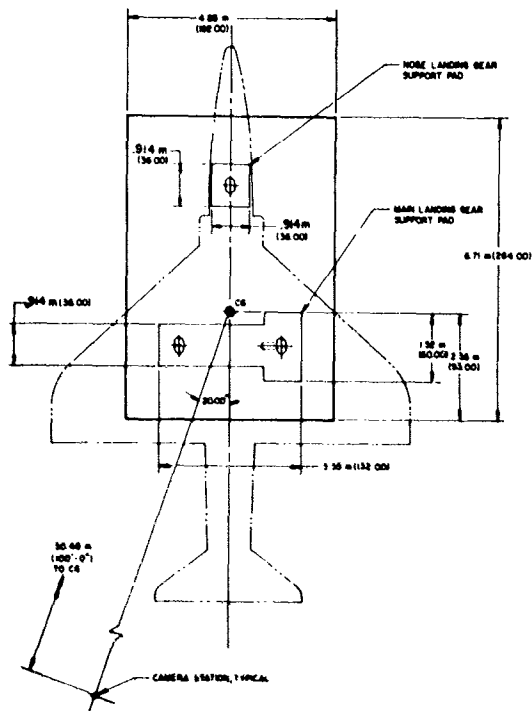
NOTE: Dimensions are in meters followed by inches in
parenthesis.

FIGURE 3. Aircraft Support Platform

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A temporary tiedown was initially applied to all aircraft
as a precaution against excessive winds. However, on the
eve of the test, a special tiedown system was installed
(Figs. 4 and 5) on two aircraft to provide biased restraint
against aircraft motion during the positive blast phase.
The third aircraft (BUNO 145062) was not tied down during
the blast in order to simulate a launch-ready condition and
to allow comparison of rigid body motions of this aircraft
with those of the secured aircraft.

INSTRUMENTATION CABLING

CAMERA POLE

MARKER STAKE

LOAD CELL NO. 1

LOAD CELL NO. 6

REMOVED AT SHOT TIME

LOAD CELL NO. 2

LOAD CELL NO. 5

LOAD CELL NO. 3 & 4

FIGURE 4. Tiedown Details

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LOAD CELL NO. 5

REMOVED AT SHOT TIME

LOAD CELL NO. 1

LOAD CELLS NO. 3 & 4

FIGURE 5. Tiedown Details

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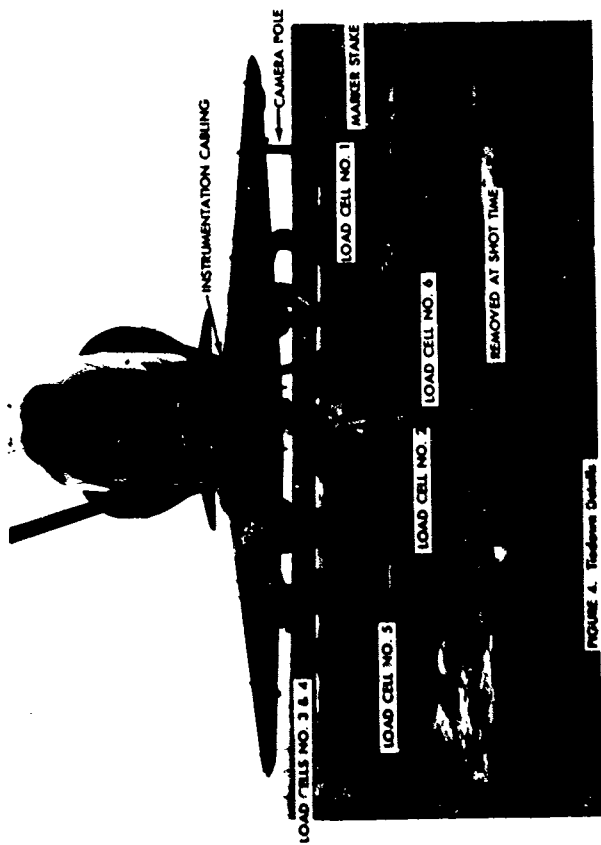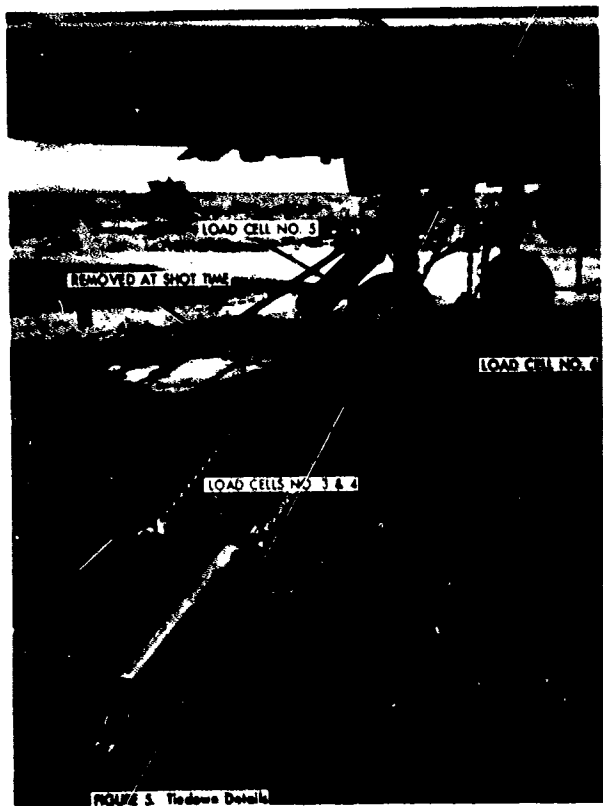
TEST INSTRUMENTATION

The overall scope of this test program called for diversified instrumentation to provide adequate data to determine two kinds of the structural response to blast, i.e., local response and rigid body response. Consequently, the instrumentation selected may be divided into two categories according to the kind of response to be monitored.

For the local structural response to overpressure, strain and deflection gages were used. Pressure gages were required to monitor transient behavior of the external pressure loading in the vicinity of the structural elements under consideration. Stress coat was also used to augment strain gage data and provide qualitative rather than quantitative information on strain patterns developed in skin panels following the shock intercept.

In the case of the structural response to gust, where time related history of the entire aircraft motion had to be obtained, strategically located motion picture cameras were used to record the aircraft rigid body motion. Also calibrated load cells were installed within the tiedown system to measure the tiedown restraint forces.

In all cases of the electronic related instrumentation, i.e., strain, deflection, and pressure gages, the data was collected on FM magnetic tape recorders located in the instrumentation trailer using 1524 m (5000 ft) of instrument cable. Three cables of 50 twisted wire pairs with overall shield were required because telemetry of data to the trailer was prohibited as a policy of the DICE THROW test director.

Individual items of the instrumentation system, the selection of the structural elements to be instrumented, and the instrumentation layout are described in separate subsections which follow.

INSTRUMENTATION DESCRIPTION

STRAIN GAGES

Micro-Measurement strain gages type 'EA-13-250UW-120 were used to obtain strain data. These gages, with 120-ohm nominal resistance, are classed as general purpose strain gages suitable for static and dynamic stress analysis

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and are capable of reliable operation within the temperature range of -75 to 250°C, over a range of ±50,000 microstrains. The self-temperature-compensation characteristics of these gages assured strain reading within ±5 percent accuracy for 0 to 100°C temperature range.

A typical strain gage circuit used a conventional Wheatstone bridge arrangement with the active gage representing one arm of the bridge. The remaining three arms were made up of wire-wound resistors specially selected for low temperature drift. Only the Wheatstone bridge was located in the aircraft; all other elements of the strain gage system were installed in the instrumentation trailer.

The gage excitation was furnished by a 10 ±.05 V power supply. The gage signal, amplified by a fixed gain dc amplifier, was recorded on a magnetic tape recorder.

This system responded with a 6-kHz frequency bandwidth and its overall accuracy was assessed at ±5 percent.

PRESSURE GAGES

Kulite pressure gages type XTH-1-190-10S were selected to obtain pressure data. This gage, specified for 0 to 138-kPa (0 to 20 psi) pressure range with an operating temperature range from -20 to +80°C, provided pressure data with the accuracy rated at ±5 percent.

A special excitation system was furnished by a 24-V battery regulated down to 20.1 V. The signal from the pressure gage was fed to a unity gain buffer amplifier located at the aircraft. This buffer amplifier was used to increase the frequency response capability. From the buffer amplifier, the signal was transmitted to the instrumentation trailer via the 50 pair cable. It was then amplified again through a dc amplifier and recorded on the magnetic tape.

This system assured overall pressure measurement accuracy to ±10 percent.

DEFLECTION GAGES

Kaman Sciences Corporation deflection gages types KD-2300-6C and -8C were used as deflection sensors. The characteristic outputs of these gages, rated at 1 percent accuracy, were 2 mV per 0.0254-mm (0.001 in.) deflection for

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-6C gage and 4 mV per 0.0254-mm deflection for -8C gage. Each gage consisted of a transducer and an oscillator package which had the operating temperature range limits of 0 to 75°C. These elements plus a ±12 V battery power source were located in the aircraft. The processed output signal was then transmitted to the instrumentation trailer where it was amplified prior to being recorded on magnetic tape.

The overall accuracy of this system was approximately ±4 percent.

STRESS COAT

As an instrumentation technique, stress coat can be successfully applied as an indicator of complex strain patterns developed in specimens under test. It is particularly useful in cases of structural components of intricate configuration for which reliable analytic methods are not available.

Stress coat, which can be easily applied to the specimen surface, cracks under load. These cracks form a pattern which can be correlated with the strains induced in the tested part, analogous to strain contours. Because of its high sensitivity to temperature changes, stress coat is best suited for tests conducted under strictly controlled environmental conditions.

For this test, stress coat was regarded strictly as a secondary instrumentation medium to augment dynamic strain gage data. Low temperature stress coat (Ref. 5) was selected because of low temperatures predicted for the test site at night.

MOTION PICTURE CAMERAS

Three 16mm Milliken model DBSS motion picture cameras were furnished to record aircraft rigid body motions following the shock wave intercept. Two cameras which had 25mm lenses were set at F-stop 2.8 and the third camera, with 13mm lens, was set at F-stop 4. The cameras were loaded with Kodak color film type EM, ASA 160, and the film speed was set at 400 frames per second.

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LOAD CELLS

The exploded view of the load cell is presented in Fig. 6 which shows the assembly. The cylindrical tube serves as a housing for an aluminum honeycomb tube core element which is used as the energy measuring structure. With tension load applied through the end fittings, the core element is subjected to a compression load. When this load exceeds a predetermined value, crushing of the core takes place and motion results. The energy absorbed in this process can be determined from the known characteristics of the core and the amount of deformation stroke produced during crushing.

SELECTION OF STRUCTURAL ELEMENTS

With the strain and deflection gages assigned as the primary instrumentation to generate the experimental data, a selection of structural elements was undertaken. From the standpoint of the experimental data quality, the criteria which governed the selection of structural elements included:

1.   Structural response preferably into the plastic range.

2.   Low probability of catastrophic failure.

3.   Applicability to computer modeling.

A broad spectrum of structural skin panels was initially contemplated to study effects of such parameters as panel size and aspect ratio, thickness and curvature, boundary conditions, and skin material. However, practical considerations which limited the choice were dictated by:

1.   Desirability for simple geometry, i.e., square or rectangular shape.

2.   Ease of modeling.

3.   Easy access to panels.

4.   Keeping panel skin thickness within a relatively narrow band to ensure post-yield response but preclude rupture.

5.   Instrumentation and recording system costs.

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FIGURE 6. Load Cell - Exploded View

The structural elements selected for instrumentation included eight skin panels, two longerons, and one fuselage frame. The panels, all made from 7075-T6 aluminum alloy were approximately rectangular. The panel locations on the airframe are shown in Fig. 7 and panel geometry data are summarized in Table 1. Panels 1 through 7 represented clamped-edge boundary conditions and an access door, panel 8, represented simply supported edge boundary conditions.

The longerons which formed the top and bottom boundaries of panel 5 together with the frame of fuselage station 277 completed the set of structural elements selected for instrumentation.

The longerons represented "L"-shape beams of uniform cross section supported at each end by adjacent fuselage frames.

The "C"-shape frame at fuselage station 277 was selected for its relatively simple cross section, basically a web with a flange on either side, forming a curved beam. Admittedly, this was a beam with cross section varying along its span, but it represented a continuous arch, symmetric about the vertical reference plane and supported by the lower longeron at either end.

INSTRUMENTATION LAYOUT

STRAIN GAGES

The strain gage instrumentation comprising 58 recording channels covered eight skin panels, two longerons, and one fuselage frame (Figs. 8 through 16). For skin panels, the gages were installed at principal locations such as the panel center, gages 1 through 4 (Fig. 8), or mid-points of panel edges, gages 27 or 47 (Fig. 11). For these locations, analytic data could be readily obtained from computer code solutions.

Clusters of strain gages were installed staggered-fashion at mid-panel locations close to the longer edge, gages 5 through 9 (Fig. 8). This arrangement was intended to provide strain gradient data in the area of the expected maximum strain variation.

20
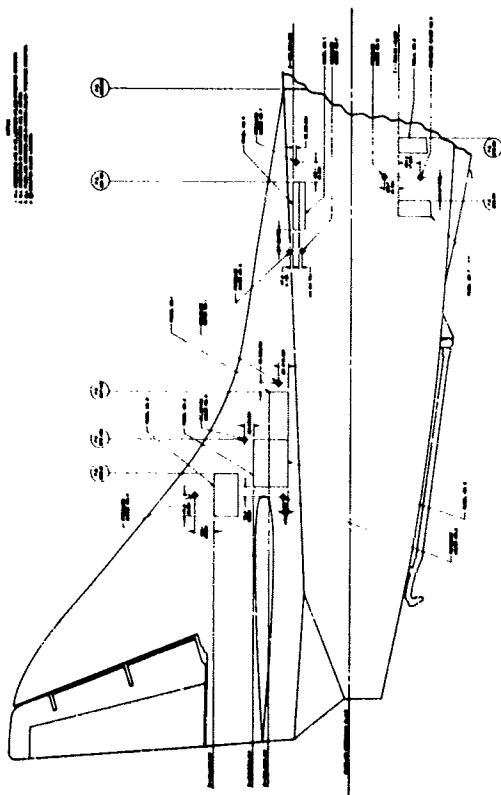
FIGURE 7. Instrumented Panels Location Diagram

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TABLE 1. Strain Gage Instrumented Panels Geometry Data.

PANEL NUMBER	RIVET LINE (REF)		EFFECTIVE SIZE		THICKNESS	RADIUS OF CURVATURE	ASPECT RATIO
	LENGTH mm (in.)	WIDTH mm (in.)	LENGTH (a) mm (in.)	WIDTH (b) mm (in.)	mm (in.)	mm (in.)	(a/b) (in.)
1	381 (15)	162 (6.375)	362 (14.25)	143 (5.625)	1.02 (.040)	00	2.53
2	381 (15)	156 (6.125)	362 (14.25)	137 (5.375)	1.02 (.040)	00	2.65
3	381 (15)	133 (5.25)	362 (14.25)	114 (4.5)	1.02 (.040)	00	3.17
4	388 (15.29)	92 (3.609)	367 (14.44)	73 (2.859)	0.81 (.032)	782 (30.8)	5.05
5	386 (15.19)	106 (4.172)	363 (14.28)	87 (3.422)	0.81 (.032)	782 (30.8)	4.17
6	225 (8.844)	120 (4.719)	200 (7.87)	98 (3.87)	0.81 (.032)	00	2.03
7	242 (9.547)	129 (5.094)	227 (8.922)	110 (4.34)	0.81 (0.32)	00	2.05
8	343 (13.5)	162 (6.375)	318 (12.5)	137 (5.375)	1.57 (.062)	00	2.32

NOTE: Numbers in parentheses denote the dimension expressed in inches.
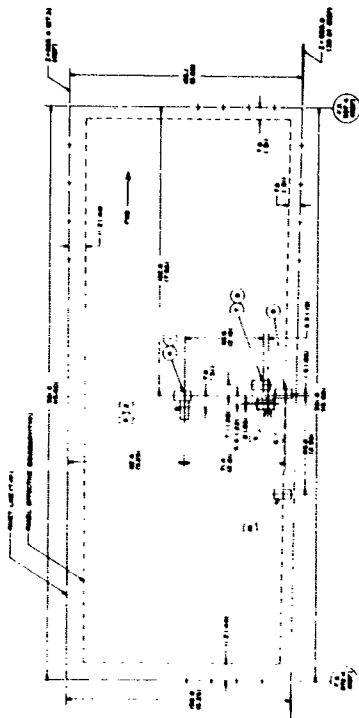
NOTE

1. All dimensions are in millimeters unless otherwise specified. Dimensions in parenthesis are in inches.

2. All fuselage stations are in inches.

3. Strain gages Nos. 3, 4, 5, 7, 9 and 10 are installed on panel outside surface.

4. Strain gages Nos. 1, 2, 6 and 8 are installed on panel inside surface.

FIGURE 8. Strain Gage Instrumentation Layout - Panel 1

23
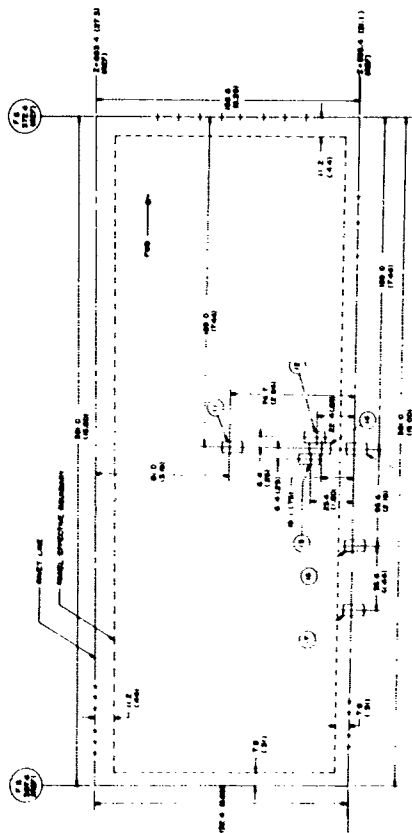
NOTE: Dimensions are in meters followed by inches in parenthesis.

FIGURE 9. Strain Gage Instrumentation Layout - Panel 2

FIGURE 10. Strain Gage Instrumentation Layout -- Panel 3

NOTE: Dimensions are in meters followed by inches in parenthesis.

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NOTE: Dimensions are in meters followed by inches in parenthesis.

FIGURE 11. Strain Gage Instrumentation Layout – Panel 4
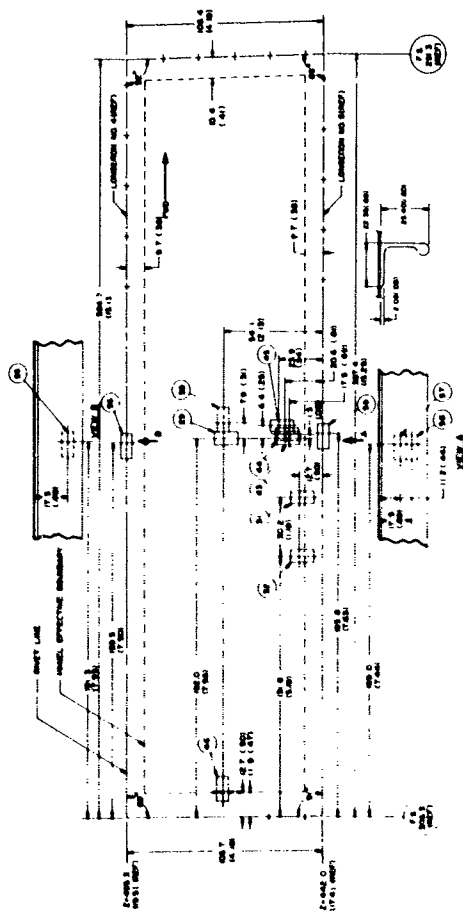
NOTE: Dimensions are in meters followed by inches in parenthesis.

FIGURE 12. Strain Gage Instrumentation Layout - Panel 5.

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NOTE: Dimensions are in meters followed by inches in parenthesis.

FIGURE 13. Strain Gage Instrumentation Layout - Panel 6

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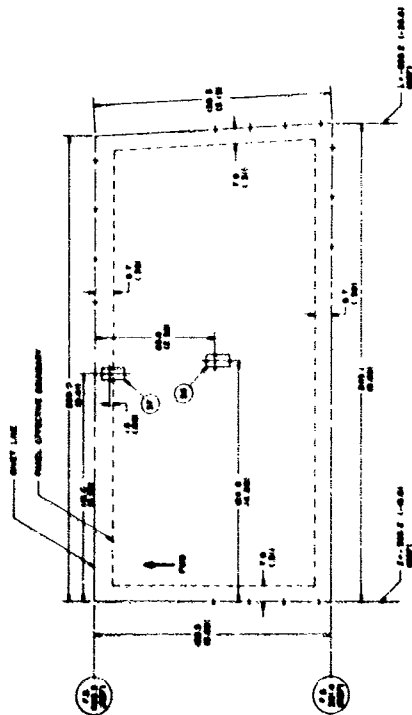
NOTE: Dimensions are in meters followed by inches in parenthesis.

FIGURE 14. Strain Gage Instrumentation Layout - Panel 3

29
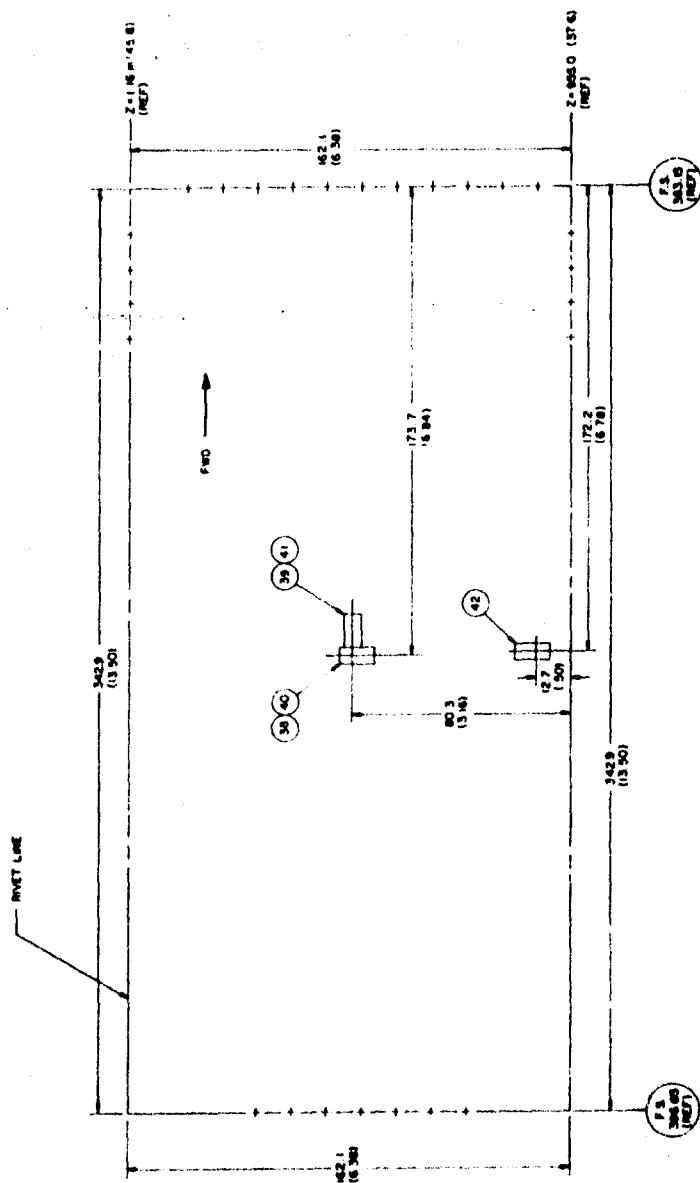
NOTE: Dimensions are in meters followed by inches in parenthesis.

FIGURE 15. Strain Gage Instrumentation Layout – Panel 8

30
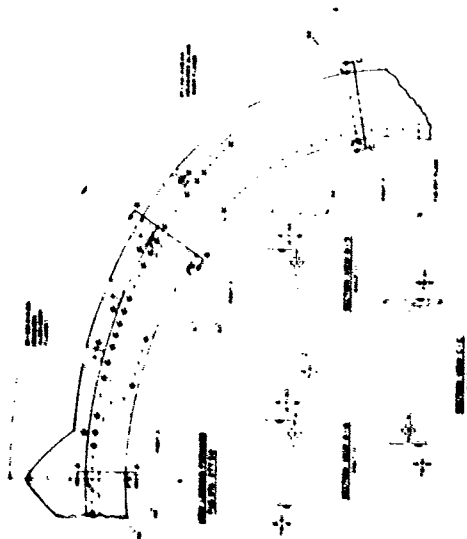
NOTE: Dimensions are in meters followed by inches in parenthesis.

FIGURE 16. Strain Gage Instrumentation Layout - Fuselage Frame Station 277.

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Sets of several strain gages, such as gages 15, 16, and 17 (Fig. 9) installed along the panel longer edge, were designed to provide data on strain behavior between the mid-point and the corner. The strain for a panel with clamped-edge boundary condition was expected to vary between a maximum for the panel at the edge mid-point and zero at the corner.

The gage locations for number 4 and 5 longerons are shown in Fig. 12. Gages 55, 57, and 58 were installed on longeron webs at mid-span between fuselage frame stations 291.3 and 206.3. Gages 56 and 59 were bonded on the fuselage skin outer surface opposite longeron flanges where the skin and the flange were assumed to act as an integral unit.

Strain gages 48 through 53 were installed in pairs at three sections along the span of the fuselage frame station 277.312 (Fig. 16). With the exception of gage 53 which was bonded to the skin outer surface, all gages were installed on the frame flanges.

DEFLECTION GAGES

The displacement measuring system consisted of three deflection gages. A pair of those gages was installed on panel 5, one to measure the deflection at the center, the other to measure deflection of the supporting substructure. The third gage was installed to measure the central deflection of panel 7.

All deflection gages were mounted in specially designed braclets which were rigidly attached to the substructure members forming panel boundaries. A typical configuration of the deflection gage assembly is shown in Fig. 17.

PRESSURE GAGES

A system of ten pressure gages was provided to monitor overpressure in the areas of instrumented skin panels (Fig. 7). Gages 1 through 4 were installed on the vertical stabilizer and monitored the area of panels 1, 2, 3, and 8 to record the pressure variation in the lateral and vertical directions. Three gages, numbers 4, 5, and 6, were assigned to the fuselage area of panels 4 and 5. The curved profile of the fuselage at that location was expected to produce significant pressure variations in the reflected pressure because of its dependence on the incident angle of the shock wave.

32
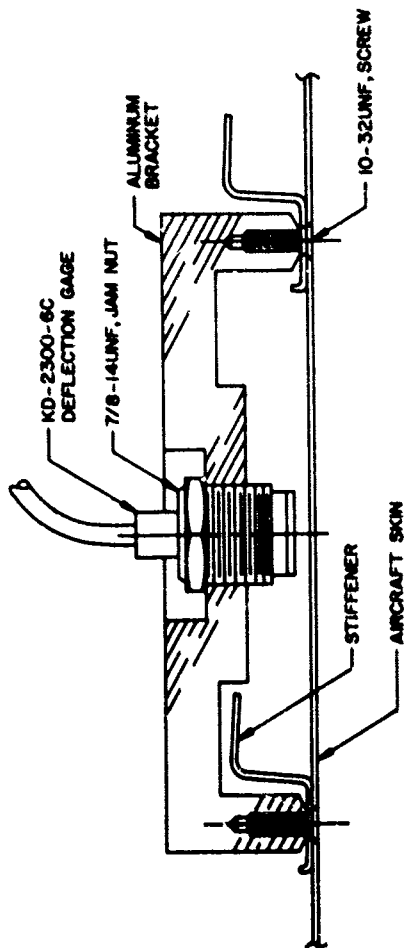
FIGURE 17. Deflection Gage Installation Diagram

Significant pressure gradient was also expected in the area of panels 6 and 7 where proximity of the wing root and fuselage intersection was likely to cause a local rise in the reflected pressure. Pressure gages 8 and 9 were located to record that pressure gradient.

Finally, gage 10 was installed inside the airframe, approximately in the middle of the fuselage, to monitor overpressure inside the structure during the diffraction

STRESS COAT

The weather forecast for the day of the test predicted the temperature drop to 5°C at the test site. Consequently, stress coat TL-500-50 (Ref. 5) was selected to minimize the hazard of crazing at low temperature because of thermal contraction. The stress coat was applied on the day preceding the test to allow the necessary curing time. This application was restricted to skin panels on aircraft BUNO 145062 and 145074 which corresponded to the strain gage instrumentation panels of aircraft BUNO 149558. Hopefully, this would allow comparison with the real-time strain gage results.

MOTION PICTURE CAMERAS

A typical camera layout given in Fig. 1 shows the camera station at 30.5 m (100 ft) from the aircraft CG with the line of sight at 20-degree angle to the aircraft longitudinal axis. Each camera was mounted on a 0.3-m (12 in.) diameter pole, 2.5 m (8.2 ft) above the ground. This approximated the height of the aircraft CG above the ground. The poles were anchored with four cables to minimize the camera motion during the shock front intercept.

The cameras equipped with 25-mm lenses were positioned to record the motion of the aircraft BUNO 149588 and 145074 and the 13-mm lens camera was used for aircraft BUNO 145062.

Scale reference was supplied by a pair of marker stakes at each aircraft. One stake was installed directly below the aft end of the engine exhaust shroud and the other was laterally displaced by approximately 1.52 m (60.0 in.) to the port side.

The measured distances between each set of marker stakes are listed below:

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AIRCRAFT IDENT. BUNO	DISTANCE
149588	1.59 m (62.75 in.)
145062	1.53 m (60.3 in.)
145074	1.53 m (60.2 in.)

LOAD CELLS

The load cells were installed as extension links within the tiedown system. The tiedown setup for aircraft BUNO 149558 and 145074 (Figs. 4 and 5) consisted of six chains attached between the aircraft and buried anchors.

This tiedown pattern was devised as the restraint to the anticipated translatory and rotational motions of the aircraft as well as the means of measuring the impulse sustained by the structure over the blast positive-phase duration.

A tether system furnished for aircraft BUNO 145062 consisted of several lengths of tiedown chain arranged in such a manner as to permit approximately 6 m (19.7 ft) of free lateral displacement before the restraint would become effective. In view of rather conflicting predictions for aircraft rigid body behavior under blast effects, this setup was designed to check the conservativeness of predictions for overturning.
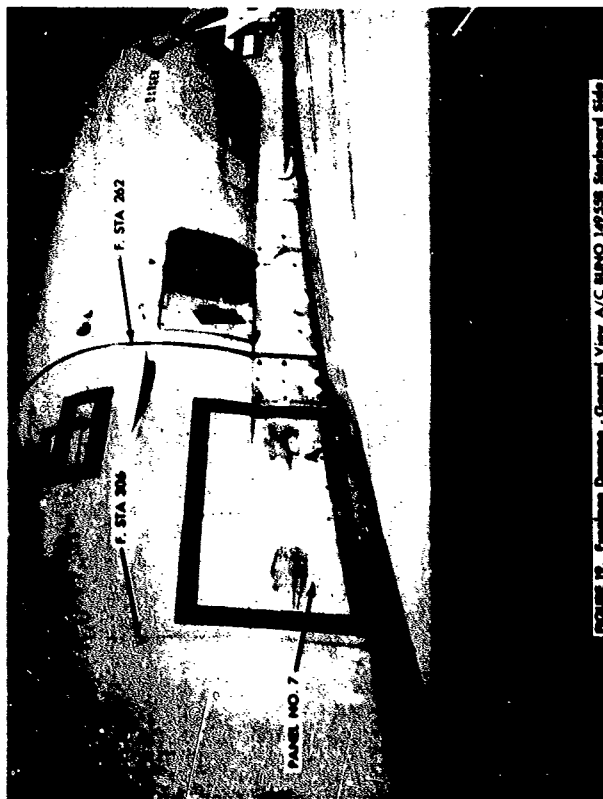
FIGURE 19. Fuselage Damage - General View A/C BUNO 149358 Starboard Side

FIGURE 20. Rudder Hinge Failure - A/C BUNO 149558

PANELS NO 1, 2, & 3 AREA

F. STA 220

F. STA 180
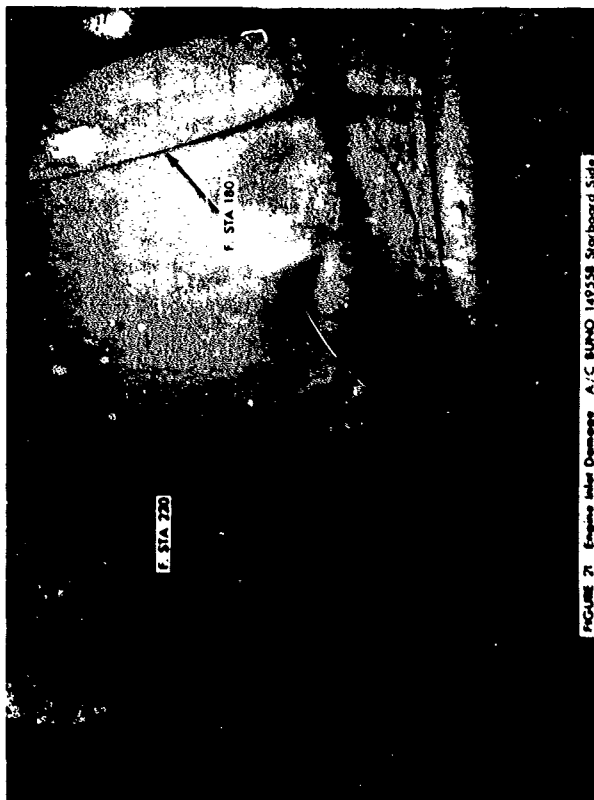
FIGURE 21 Engine Inlet Damage A/C BUNO 149558 Starboard Side
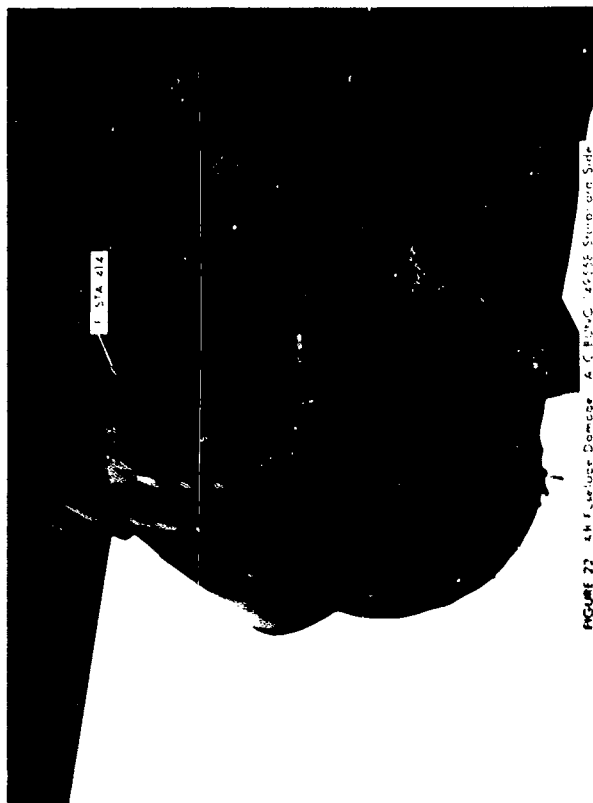
FIGURE 22 LH Fuselage Damage - A/C P/N-C 46558 Structure Side

FIGURE 22. Fuselage Damage Aft of F. STA 305 - A/C BUNO 149253 Starboard Side

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F. STA 306

F. STA 262

PANELS NO. 6 & 7

FIGURE 34. Mid - Fuselage Damage - A/C BUNO 149558 Starboard Side
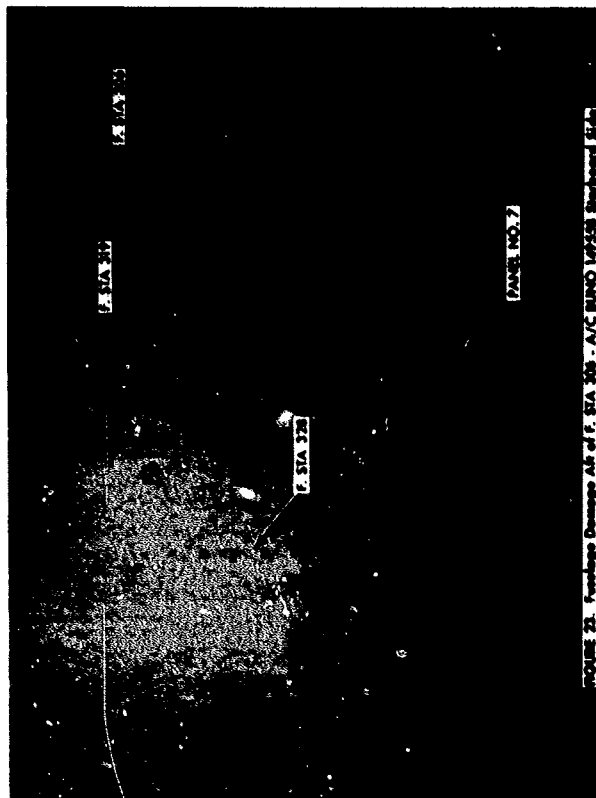
TEST RESULTS

GENERAL

This section of the report is devoted to a detailed
description of damage and a presentation of selected experi-
mental data obtained from the instrumentation installed in
aircraft BUNO 149558, which was exposed to 41.4-kPa (6 psi)
peak free-field overpressure. A complete presentation of
the experimental data is included in Volume II of NWEF
Report 1145 (Ref. 6).

As mentioned previously, two aircraft were located
375 m (1230 ft) from the GZ. At this range, the peak free-
field overpressure was approximately 41.4 kPa (6 psi). The
third aircraft, located 311 m (1020 ft) from the GZ experi-
enced approximately 62.1-kPa (9 psi) peak free-field over-
pressure. Accordingly, the degree of damage caused by the
blast was expected to differ significantly in those two
cases. An inspection following the test revealed relatively
light damage sustained by the aircraft farther away from the
GZ whereas the aircraft closer to the GZ ruptured in a
catastrophic failure (Fig. 18). This leads to an immediate
and rather significant observation that within very narrow
limits of the distance from the GZ, approximately 61 m
(200 ft) in this case, the damage sustained by the aircraft
varied from slight to catastrophic.

DAMAGE DESCRIPTION

AIRCRAFT BUNO 149558

The posttest inspection of the exterior of this air-
craft, which experienced 41.4-kPa (6 psi) peak free-field
overpressure, revealed widely scattered local damage on the
starboard side (i.e., the side exposed to blast), ranging
from buckling of the skin panels to rupture of the skin
(Fig. 19), failure of the landing gear doors, and loss of
the rudder (Fig. 20). Details of skin buckling are illus-
trated in Figs. 21 and 22 which show the engine inlet at
fuselage STA 175 and the fuselage region aft of STA 400,
respectively, where the skin material was 0.81-mm (0.032 in.)
thick. Figure 23 shows the fuselage in the region aft of
Sta 306 where extensive rivet failures were observed.

More severe damage was sustained in the vicinity of the
forward and aft fuselage joint, STA 262. Just aft of that
station (Fig. 24), a skin rupture initiated at fuselage

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STA 277, approximately 220 mm (8.5 in.) above the wing
surface, and proceeded diagonally towards the wing and then
along the lower longeron for about 355 mm (14 in.). From
this area of panel rupture, a severe skin buckle radiated
upwards crossing several panel boundaries. Also shown in
Fig. 24, forward of STA 262, is the failure of a non-structu-
ral panel which ruptured at its lower boundary and was
forced inward under the overpressure loading.

Regarding the landing gear door damage, it can be
generally stated that the pretest arrangement of fastening
these doors by means of cables and struts to compensate for
the deactivated hydraulic system failed completely. In
addition, failures of the door structure were incurred as
exemplified by the rupture of the nose landing gear retrac-
tion fitting and the failure of the main landing gear port-
side door hinge.

Slight buckles were observed on the port side (i.e.,
side not exposed to the blast) on the engine inlet duct and
mid-fuselage STA 300.

Moderate damage of substructure was noted on the
starboard side of the aircraft at the mid-fuselage region
between Stations 236 and 342. The damage included sheared
fasteners at the joint of such major components as a former
and the lower longeron (Fig. 25), buckles and cracks in the
stiffener or former flanges (Fig. 26), and a complete fail-
ure of a former cross section (Fig. 27). Significantly,
fastener holes were invariably present at the crack or
rupture locations which implies that considerable strength
degradation associated with the presence of a fastener hole
may lead to premature failure of the substructure element.

<u>AIRCRAFT BUNO 145062</u>

This aircraft, also subjected to 41.4-kPa (6 psi) peak
free-field overpressure, sustained, in general, an identical
type of damage as the first aircraft. However, a distinct
evidence of greater severity of the damage was apparent.
This is illustrated in Fig. 28, where the buckles of the aft
fuselage region form a much more clearly defined pattern, in
Fig. 29 which shows more extensive skin rupture in the
fuselage frame Sta 277 area, and in Fig. 30 which shows
flange failure in the frame itself. Similarly, Fig. 31
shows fuselage STA 320 area with the skin ruptured whereas
only fastener failures were observed for the other aircraft.
Also, more severe landing gear door failures were observed.

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FRAME F, STA 277.
FRAME CAP DISLOCATED.
THROUGH FASTENER FAILURE

LOWER LONGERON

Figure 33. Frame Joint to Lower Longeron Failure - A/C BUNO 149558 Starboard Side

STIFFENER F. STA 314

STIFFENER FLANGE & WEB CRACK
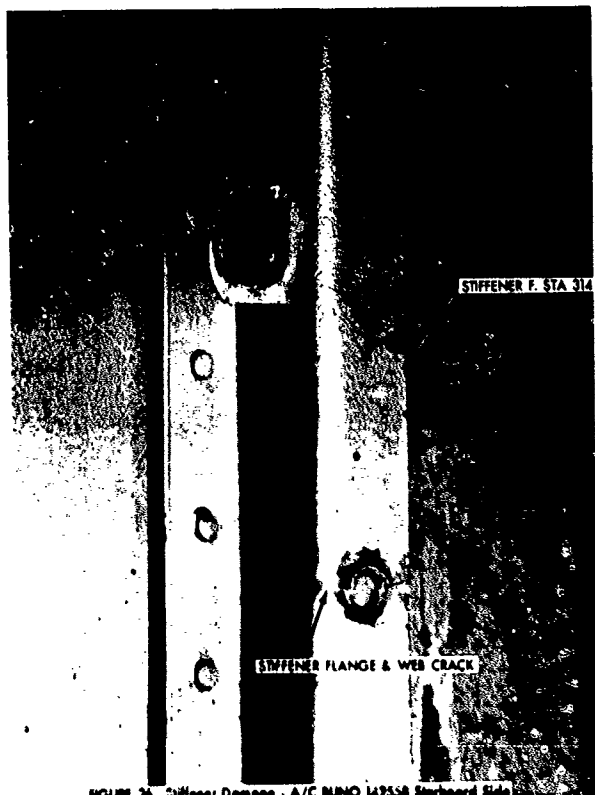
FIGURE 26 Stiffener Damage - A/C BUNO 147558 Starboard Side

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RUPTURED FORMER
ENTIRE CROSS SECTION

REINFORCING DOUBLER AT
WATER LINE -15

FIGURE 27. Former Damage - A/C BUNO 149165

F. STA 414

FIGURE 28. Aft Fuselage Damage - A/C BUNO 142062 Starboard Side

F. STA 277

WING - FUSELAGE
INTERSECTION
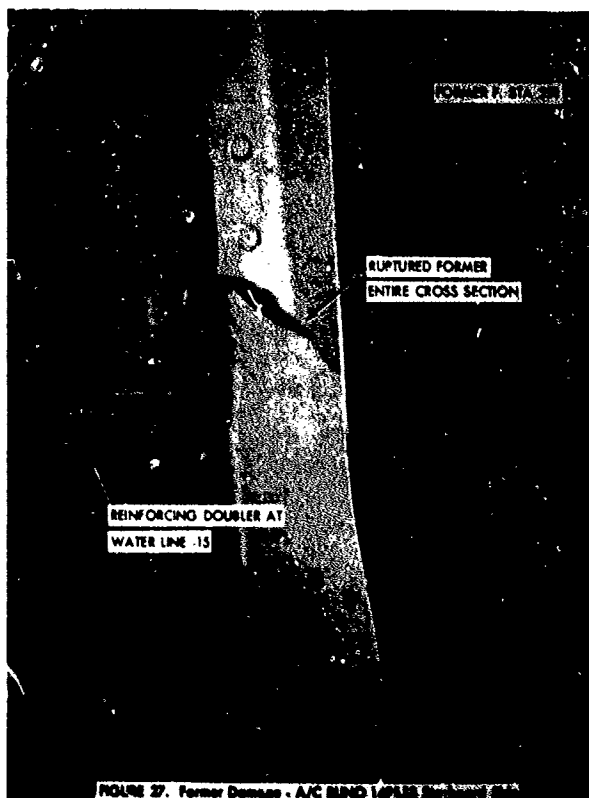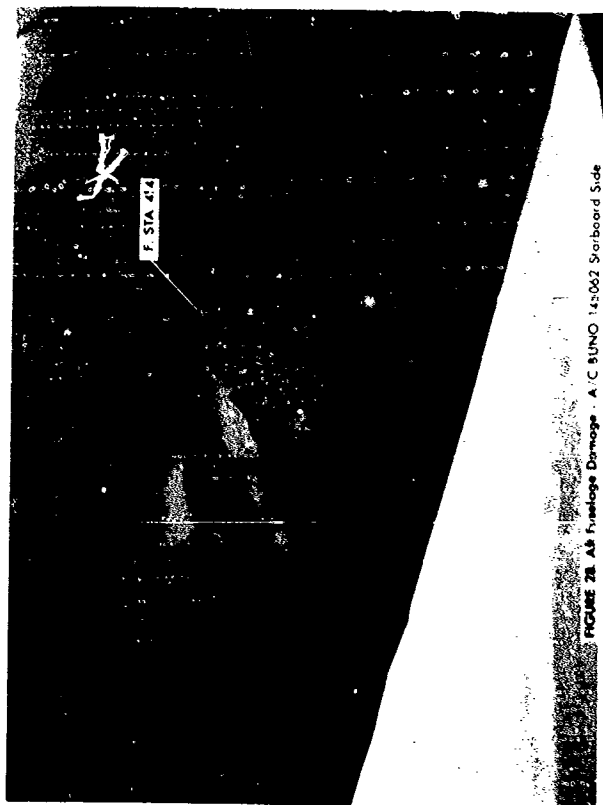
FIGURE 29 Mid - Fuselage Damage - A/C BUNO 145062 Starboard Side

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FIGURE 31. A3 Fuselage Damage - A/C BUNO 143052 Starboard Side

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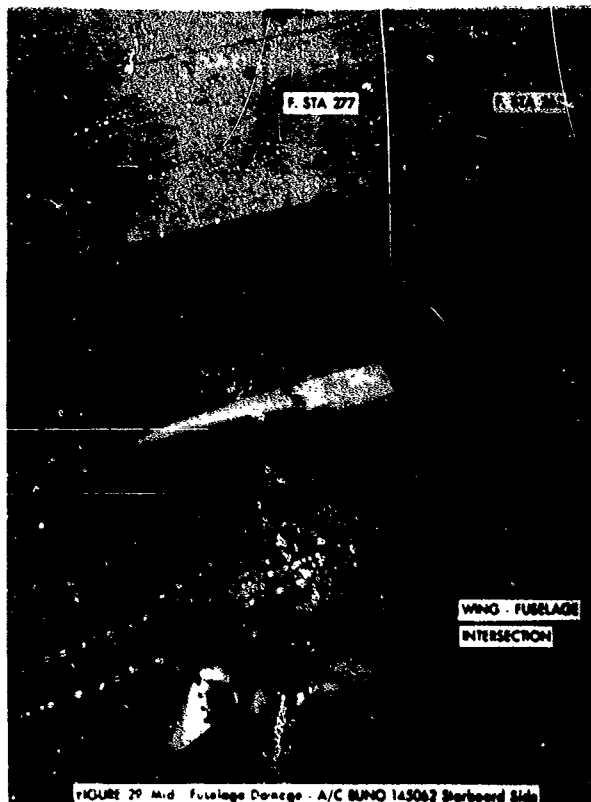
AIRCRAFT BUNO 145074

The devastating effects of the blast were clearly
demonstrated (Fig. 18) for this aircraft which suffered
catastrophic failure under 62.1-kPa (9 psi) peak free-field
overpressure. The aft fuselage section between Stations 262
and 342 was virtually demolished, presumably under overpres-
sure loads. The rupture of lower longerons, which constitute
the primary structural members in the axial direction,
resulted in the empennage section breaking away from the
rest of the airframe.

The catastrophic nature of the damage was also apparent
from complete disintegration of the canopy (Fig. 32) as well
as from the extensive skin rupture and extreme severity of
buckling patterns in the engine inlet duct structure (Fig. 33).

The failures of the primary substructure members are
illustrated in Figs. 34 through 36. The appearance of
ruptured cross sections suggests tensile failures for the
starboard upper (Fig. 34) and lower (Fig. 35) longeron
members. The port lower longeron (Fig. 36) exhibits a
compressive failure characteristic.

DAMAGE ASSESSMENT

Considering the blast characteristics at the two
principal aircraft locations shown in Table 2, it is not
surprising that the extent of damage sustained by the air-
craft differed significantly in each case. However, based
on the pretest predictions, it was assumed that the aircraft
exposed to 41.4-kPa (6 psi) overpressure might sustain
local, permanent-set type damage of the weakest panels or
substructure elements, but no failures in the form of panel
skin rupture or fracture of an entire former cross section
were anticipated (Figs. 24 and 27). Similarly, for the
aircraft exposed to 62.1-kPa (9 psi) overpressure, even
though a possibility of failure was not ecompletely ruled
out, it was presumed that the damage, however extensive,
would not precipitate the catastrophic failure which actually
occurred (Fig. 18).

FIGURE 32 Canopy Failure A C BUNO 145074

FIGURE 33. Engine Inlet Damage - A/C BUNO 14507.

F. STA 120

F. STA 72

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LONGERON NO 5

F STA 277

FIGURE 34. Longeron No 5 Rupture - A/C BUNO 145074

LOWER LONGERON RAPTURED
CROSS SECTION
F. STA 340

FIGURE 35. Lower Longeron Failure, Starboard Side - A/C BUNO 148074

LOWER LONGERON RUPTURED
CROSS SECTION
F. STA 243
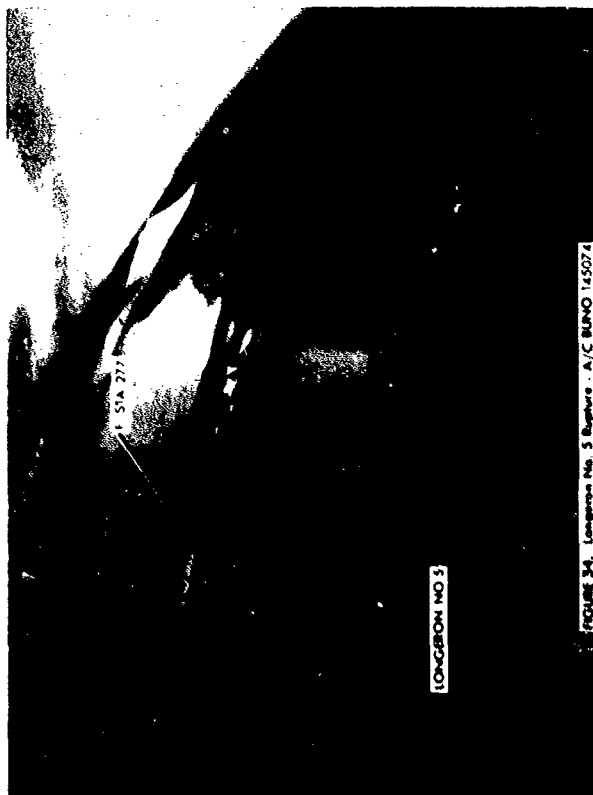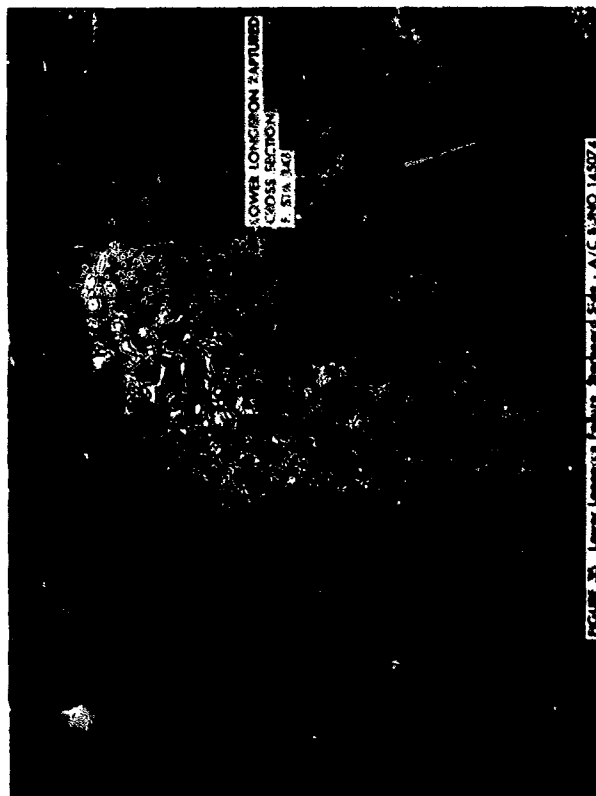
FIGURE 36 Lower Longeron Failure, Port Side - A/C BUNO 145074

TABLE 2. Predicted Free-Field Airblast Parameters.

DISTANCE FROM GZ	OVERPRESSURE	OVERPRESSURE IMPULSE	HORIZ. DYN. OVERPRESSURE	HORIZ. DYN. OVERPRESSURE IMPULSE
m	kPa	kPa·ms	kPa	kPa·ms
311	62.1	4588	13.8	799
375	42.4	3792	6.8	455

The fact that structural failures did occur can be partially attributed to the strength degradation of the aircraft structure, resulting from extensive in-service use, as well as to the synergistic effect of the overpressure and gust interaction. These were not considered in the structural integrity assessment by analytic methods.

The overall damage sustained by the two aircraft located 375 m (1020 ft) from the GZ can be divided into three categories:

1. Loss of the rudder.

2. Buckling of the skin panels.

3. Failure of substructure elements and associated skin panel rupture.

The side-on intercept of the aircraft by the blast represented the most severe orientation for the vertical stabilizer including the rudder. The rudder structural integrity was not subjected to pretest analytic prediction. Therefore, comments regarding the loss of the rudder resulting from hinge failure (Fig. 20) are limited to stating that the loads associated with 41.4-kPa (6 psi) free-field peak overpressure exceeded the hinge strength at some point and resulted in failure.

The occurrence of moderate skin buckling was primarily restricted to the engine inlet (Fig. 21) and the fuselage section aft of STA 414 (Fig. 22) where panels with thinnest skin, 0.81 mm (0.032 in.), were located. Characteristically, these panels were susceptible to early buckling because they were long with an aspect ratio approaching 6.0. This type

of damage was associated with only a minor substructure deformation as suggested by the appearance of the engine inlet area, where several rivet failures were also observed.

The severity of the buckling pattern in the aft fuselage area differed noticeably for the two aircraft located 375 m (1230 ft) from the GZ. The more severe buckling in the case of aircraft BUNO 145062 (Fig. 28), as compared to aircraft BUNO 1459558, can be explained by potentially different strength characteristics of the individual aircraft. However, it is possible that a slight variation in the blast intensity had occurred locally and was sufficient to produce discernible difference in the damage level of those two aircraft. Also, variations in the tiedown arrangement for the two aircraft might have affected the gust response and produced differences in the aft area buckling patterns.

The most severe damage was obtained in the mid-fuselage area between bulkheads STA 262 and 306 (Fig. 19), immediately above the wing-fuselage intersection. The skin rupture in panel 6 (Fig. 13) was associated with failures of numerous substructure elements. Ignoring for a moment the substructure failures, it is difficult to explain the skin rupture on the basis of the panel geometry relative to other panels within the airframe. Admittedly, the ruptured panel skin (Fig. 24) was only 0.81 mm (0.032 in.) thick, but its length, 200 mm (7.87 in.), was considerably less than the 363-mm (14.28 in.) long panel 5 of idential skin thickness where no signs of damage were found.

It may be suggested that, at this location, where the wing and fuselage met, the local shock reflection produced higher overpressure, but that alone could not account for the skin rupture. A much more plausible explanation of the skin rupture in this area is that it occurred in conjunction with the substructure failure. The structural distortion produced by the former failure at STA 291 (Fig. 27) and the fastener failures at the joint of STA 277 former to the lower longeron (Fig. 25) had a compounding effect which led to the skin rupture.

This category of damage was also more severe in aircraft BUNO 145062 (Fig. 29) as compared to BUNO 149558 (Fig. 24). The former shows the skin rupture extending upwards towards the top of the fuselage, while the latter failure is contained within the lower fuselage area. A similar situation

prevailed in the aft fuselage region of STA 319 and 328 where the fastener failure and slight damage of the subs structure for aircraft BUNO 149558 (Fig. 23) were contrasted with a more extensive damage including the skin rupture for aircraft BUNO 145062 (Fig. 31). The explanation given earlier for the panel buckling patterns is considered equally applicable in this case.

The assessment of the catastrophic failure of aircraft BUNO 145074 can best be made in the context of the blast parameter data (Table 2) and the nature of damage sustained by the aircraft located farther away from the GZ. A comparison of the blast parameters shows 50 to 100 percent higher intensity in overpressure and dynamic pressure, respectively, for the 311-m (1020 ft) location over the 375-m (1230 ft) location from the GZ.

The failure sequence as reconstructed from the post-test examination of the aircraft was as follows. The failure initiated in the vicinity of STA 277 was produced by the overpressure loads just as in the case of the aircraft located 375 m (1270 ft) from the GZ. Naturally, with the 50 percent higher overpressure, this failure resulted in the skin rupture extending possibly from the lower longeron to the top of the fuselage. This was accompanied by extensive substructure failure over the entire fuselage segment between bulkheads STA 262 and 343 which was crushed into the fuselage body. The main skeletal members of the fuselage, the longerons, appeared to have remained intact to this point, but the progressive loss of skin strength was sufficient to complete the rupturing process of the entire fuselage cross section under the subsequent gust loads. This is substantiated by a tension fracture appearance of the starboard longerons, i.e., those towards the GZ (Figs. 34 and 35), and a compression fracture of the portside lower longeron (Fig. 35). It is interesting to note that the initial failure of the upper longeron occurred at STA 277 (Fig. 33) which coincides with the most severe substructure damage to the aircraft located 375 m (1230 ft) from the GZ. Both lower longerons (failure at STA 343) contained an apparent weakness in the form of cutout holes.

INSTRUMENTATION DATA

The experimental data in their entirety are compiled in Volume II of NWEF Report 1145 (Ref. 6). This includes the data obtained from NWEF Projects implemented in both PRE-DICE THROW II and DICE THROW Events.

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However, for the purpose of correlation, pertinent
experimental data are presented graphically in this volume
in conjunction with the analytically predicted data for
overpressure, strain, and deflection. In addition, selected
strain gage data plots are included to illustrate relevant
subject material covered in the discussions. References to
these experimental data are made within the text as applicable.

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ANALYTIC PROCEDURES

The analytic treatment of the aircraft structural response to overpressure provided in this report is based on three overpressure analysis methods formulated in the "Handbook for Analysis of Nuclear Weapon Effects on Aircraft" (Ref. 2). The complexities of the structural modeling and the nuclear blast representation vary with each method. Presumably, the accuracy of the analytic predictions vary accordingly.

METHOD 1

The least complex technique, Method 1, is based on two major assumptions:

1. Overpressure damage to an aircraft is the same for all aircraft of a given generic type.

2. The preblast atmosphere is homogeneous, having characteristics associated with aircraft altitude.

The method employs predetermined, critical sure-safe or sure-kill overpressure levels depending on the type of aircraft under consideration. Then, given the weapon yield, this method can be readily used to determine critical ranges and obtain plots of sure-safe and/or sure-kill envelopes.

For A-4C aircraft, which is classed in the fighter or fighter-bomber category, the method prescribes the following critical pressures:

1. 17.24 kPa (2.5 psi) for sure-safe condition

2. 137.9 kPa (20.0 psi) for sure-kill condition

Sure-safe and sure-kill envelopes were constructed based on the scenario data:

Aircraft altitude HZ = 1.2 km (4000 ft)

Ground altitude   HG = 1.2 km (4000 ft)

Equivalent yield   W = 4.2 TJ (1 KT)

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A step-by-step procedure was applied using furnished graphical and tabular data which included special provisions to account for the ground reflection effects.

METHOD 2

Method 2 is based on the premise that an aircraft subjected to an overpressure loading can experience structural damage in several ways. Skin panels may yield or rupture, while longerous, stringers and frames may fail by compressive yielding or local buckling. Since the fuselage is generally the most susceptible to these types of damage, it is explicitly considered for overpressure effects according to the analytic procedure defined by Method 2.

The method is governed by the following assumptions:

1. The skin panel is thin, flat, and rectangular with edges restrained against rotations and deflections, i.e., clamped.

2. The stringer or longeron supports one-half the pressure load acting on adjacent panels.

3. The ends of a stringer or longeron are fully restrained against deflection and rotation.

4. The frame is circular and has constant cross-section dimensions.

5. The frame supports one-half the pressure load acting on the fuselage between the frame preceding and the frame following the one being considered.

6. The pressure loading is transferred to the frame by the skin and stringers and local buckling of flanges or webs occurs before an overall instability of the frame develops.

7. Initial stresses due to preblast flight loads are negligible.

8. The preblast atmosphere is homogeneous, having characteristics associated with the aircraft altitude.

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Regardless of the structural element under consideration, the analytic procedure consists of two basic phases. In the first phase, the static critical overpressure to produce the onset of damage is determined based on the material yield stress allowables. For the skin panels, the allowable stress corresponds to the tensile yield of the material. For the stringers, longerons and frames, the allowables are defined in terms of the elastic buckling (crippling) stress or the compressive yield stress, whichever is less.

In the second phase, a dynamic factor is computed based on the natural frequency of the structural element under consideration as well as on the time duration of the reflected blast wave and the reflection factor. The resultant critical overpressure value is then obtained as the critical static overpressure divided by the dynamic factor.

The sure-safe condition is defined according to the minimum critical overpressure computed for the weakest panel, stringer, longeron, or frame. The sure-kill condition is based on the critical overpressure determined for the strongest longeron with the cross-section characteristics and spacing corresponding to a fuselage station near the forward end of the tail cone.

The analysis of A-4C aircraft in accordance with Method 2 was accomplished using the OVPR2 code, which is structured to follow the iterative computational procedure developed for this method (Ref. 2). The required input data for OVPR2 solutions were chosen to ensure a basis for correlation between the analytic and the experimental data. The structural elements modeled correspond to those instrumented in the DICE THROW experiment. Skin panels 1, 5, and 6, in Figs. 8, 12, and 13, respectively, were represented in this manner to conform to the input format for an OVPR2 solution. Panel 1 was entered as the side panel, panel 5 as the top panel, and panel 6 as the bottom panel; see Table 3.

The analytic representation of the longeron element for the sure-safe condition was based on longeron 5 configuration, Fig. 12. As shown in Table 4, the critical overpressure is a function of the angle of incidence. This angle was assessed at 32 degrees at the longeron 5 location.

Following the Method 2 criteria, the analysis for the sure-kill condition was based on the section characteristics of the lower longeron, i.e., the strongest longeron, at

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TABLE 5. OVPR 2 Solution for Panels 1, 5, and 6 -
Sure-Safe Listing of Input and Output Data.

OVERPRESSURE ANALYSIS, METHOD 2, SECTION C1.2
AIRCRAFT IDENTIFICATION - OVERPRESSURE ANALYSIS, A-4C AIRCRAFT, DICETHROW
THE CONDITION IS SURE-SAFE

SINGLE LAYER PANELS
THE AIRCRAFT ALTITUDE IS   4900.0 FEET
THE WEAPON YIELD IS   1.00 KT
THE HEIGHT OF THE GROUND ABOVE SEA LEVEL IS   4000.00 FEET
THE AIRCRAFT IS PARKED

	TOP PANEL	SIDE PANEL	BOTTOM PANEL
PANEL WIDTH BETWEEN SUPPORTING STRINGERS (IN)	3.4	5.6	3.9
PANEL LENGTH BETWEEN SUPPORTING FRAMES (IN)	14.6	14.2	7.9
PANEL THICKNESS (IN)	.032	.060	.032
RADIUS OF FUSELAGE (FT)	2.6	3.0	2.6
MODULUS OF ELASTICITY (PSI)	.104E-08	.104E-08	.104E-08
TENSILE YIELD STRESS (PSI)	.630E-05	.630E-05	.630E-05
INTERNAL BAY PRESSURIZATION =	0.000 PSI		
WEIGHT DENSITY (LBS/CU IN)	.100	.100	.100

CRITICAL VALUES OF LOCAL OVERPRESSURE FOR THE SKIN PANEL

THETA (DEGREES)	TOP PANEL	SIDE PANEL	BOTTOM PANEL
0.0	10.50	7.83	9.15
10.0	8.71	6.57	7.60
20.0	7.56	5.75	6.62
30.0	6.77	5.15	5.92
40.0	6.24	4.73	5.44
50.0	5.65	4.42	5.09
60.0	5.55	4.18	4.83
70.0	5.42	4.10	4.77
80.0	5.29	4.02	4.62
90.0	5.17	3.94	4.52

TABLE 4. OVPR 2 Solution for Longeron 5 - Sure-Safe Condition
Listing of Input and Output Data.

fuselage station 387, Fig. 37. OVPR2 input data for the
lower longeron together with the analytic results are pre-
sented in Table 5.

The frame analysis restricted, as in the case of skin
panels, to the sure-safe condition was based on the fuselage
frame station data presented in Table 6 which also includes
the analytic results.

METHOD 3

By far the most sophisticated of the three methods,
Method 3, offers detailed and diversified modeling of air-
craft structural elements. It also includes a comprehensive
blast representation based on a state-of-the-art airblast
model and permits completely arbitrary selection of the
scenarios defining the spatial relation of the aircraft with
respect to the detonation point. In its latest version,
Method 3 contains a special provision for a point by point
representation of the blast characteristics on the local
level (i.e., pertaining directly to the structural element
analyzed) which can be advantageously applied to simulate
actual experimental data.

NOVA-2 (Nuclear Overpressure Vulnerability Analysis,
Version 2), Ref. 7, the computer program for Method 3,
provides a technique for predicting the elastic and elastic-
plastic response of aircraft structural elements to tran-
sient pressure loads associated with the blast wave from a
nuclear explosion.

The program consists of three distinct routines, NOVA,
DEPROB (Dynamic Elastic Plastic Response of Beams), and
DEPROP (Dynamic Elastic Plastic Response of Panels), written
in FORTRAN IV language.

The NOVA routine is the master routine which controls
the logic of the overall program. It contains the sub-
routines for (1) predicting the aerodynamic flight loads and
the blast pressure loads that are applied to the lifting
surfaces and fuselage during subsonic and supersonic flight
and (2) determining the slant range at which a structural
element incurs damage which has been specified on a probabi-
listic basis.

The DEPROB routine provides the response of aircraft
structure such as stringers, longerons, frames, ribs, and
conical or cylindrical radomes which can be represented by

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FUSELAGE
CONTOUR

3.18(.13)

X = 10.00

Z = -20.00
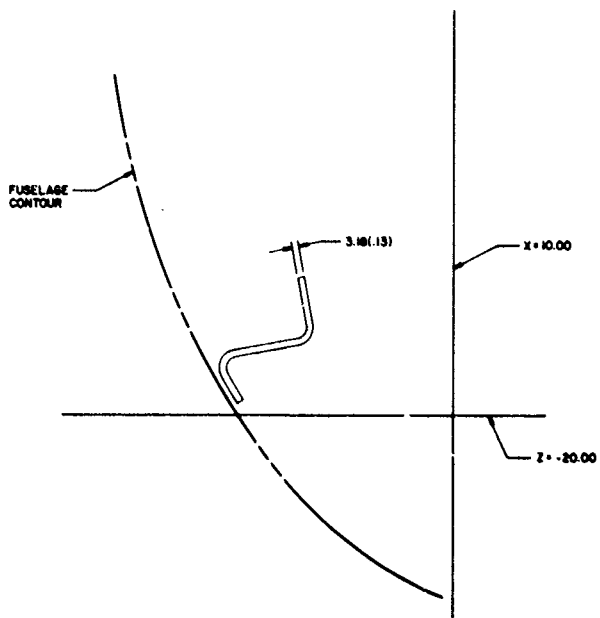
FIGURE 37.   Lower Longeron Configuration Diagram.

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TABLE 5. OVPR 2 Solution for Lower Longeron - Sure-Kill Condition Listing of Input and Output Data.

OVERPRESSURE ANALYSIS-. METHOD 2. SECTION C1.2
AIRCRAFT IDENTIFICATION - OVERPRESSURE ANALYSIS. A-4C AIRCRAFT. DICETHROW
THE CONDITION IS SURE-KILL

SINGLE LAYER PANELS
THE AIRCRAFT ALTITUDE IS 4000.0 FEET
THE WEAPON YIELD IS 1.00 KT
THE HEIGHT OF THE GROUND ABOVE SEA LEVEL IS 4000.00 FEET
THE AIRCRAFT IS MARKED

	TOP LONGERON	BOTTOM LONGERON
DISTANCE TO ADJACENT STRINGER ON EITHER SIDE (IN)	27.8	27.8
	27.8	27.8
LONGERON LENGTH BETWEEN FRAMES (IN)	14.5	14.5
THICKNESS OF INNER FLANGE (IN)	.125	.125
WIDTH OF INNER FLANGE (IN)	1.25	1.25
RADIUS OF FUSELAGE (FT)	1.6	1.6
CROSS-SECTIONAL AREA (SQ IN)	.5880	.5880
DISTANCE FROM INNERMOST EDGE TO CENTROID (IN)	1.34	1.35
AREA MOMENT OF INERTIA (IN**4)	.187	.187
MODULUS OF ELASTICITY (PSI)	.106E+08	.106E+08
COMPRESSIVE YIELD STRESS (PSI)	.700E+05	.700E+05
WEIGHT DENSITY (LBS/CU IN)	.100	.100

THE BUCKLING COEFFICIENT FOR INNER FLANGE 1 IS K = 1.25
THE BUCKLING COEFFICIENT FOR INNER FLANGE 2 IS K = 1.25

CRITICAL VALUES OF LOCAL OVERPRESSURE FOR LONGERONS

THETA (DEGREES)	TOP LONGERON	SIDE LONGERON	BOTTOM LONGERON
0.0	15.04	15.04	14.99
10.0	12.27	12.27	12.22
20.0	10.60	10.60	10.56
30.0	9.45	9.45	9.42
40.0	8.79	8.79	8.76
50.0	8.24	8.24	8.24
60.0	7.69	7.69	7.86
70.0	7.69	7.69	7.66
80.0	7.69	7.69	7.66
90.0	7.30	7.30	7.27

TABLE 6. OVPR 2 Solution for Fuselage Frame - Sure-Safe Condition
Listing of Input and Output Data.

CRITICAL FRAME

DISTANCE TO ADJACENT FRAMES	15.3
ON EITHER SIDE (IN)	14.3
THICKNESS OF INNER FLANGE (IN)	.050
WIDTH OF INNER FLANGE (IN)	.20
RADIUS OF FUSELAGE (FT)	2.0
CROSS-SECTIONAL AREA (SQ IN)	.3162
DISTANCE FROM INNERMOST EDGE TO CENTROID (IN)	2.33
AREA MOMENT OF INERTIA (IN***)	.435

MODULUS OF ELASTICITY (PSI)	.10+E+08
COMPRESSIVE YIELD STRESS (PSI)	.640E+05
WEIGHT DENSITY (LBS/CU IN)	.100
THE BUCKLING COEFFICIENT FOR THE INNER FLANGE	15 1.25

CRITICAL VALUES OF LOCAL OVERPRESSURE FOR FRAMES

THETA (DEGREES)	TOP FRAME	SIDE FRAME	BOTTOM FRAME
0.0	5.76	5.76	5.78
10.0	5.04	5.04	5.04
20.0	4.52	4.52	4.52
30.0	4.11	4.11	4.11
40.0	3.80	3.80	3.80
50.0	3.55	3.55	3.55
60.0	3.36	3.36	3.35
70.0	3.24	3.24	3.24
80.0	3.23	3.23	3.23
90.0	3.18	3.18	3.18

an annular cross section. The method of analysis used in this routine applies to beams which can be modeled in one dimension by a series of discrete masses interconnected by weightless springs. It features the ability to analyze elements with variable cross section, both simply supported and free edge conditions, an improved elastic-plastic stress-strain model, and the inclusion of rib buckling as a failure mechanism.

The DEPROP routine provides the response of aircraft skin panels, canopies, and radomes that can be approximated by a cylindrical panel. The linear elastic option applies to single and multilayered panels of isotropic or ortho-tropic material; and the elastic-plastic option applies to single-layered panels of isotropic material. DEPROP includes (1) symmetric or nonsymmetric combinations of clamped or simply supported edge constraints, (2) a much improved elastic-plastic stress-strain model, and (3) improved overall accuracy.

Method 3 offers individual treatment for a variety of structural elements through specified input data which call for detailed modeling of an individual element under conside-ration and the surrounding structure. This enables the determination of the local overpressure conditions in con-junction with the blast model incorporated within the program.

When local overpressure data are available, e.g., experimental data, the program computes transient response of that individual element based on the supplied overpressure data.

A special provision of the program allows a panel response solution through a beam representation. This solution yields results of acceptable accuracy and offers a considerable saving in the computer time which may be one-tenth of that required for a corresponding regular panel solution.

The A-4C aircraft analysis using NOVA-2 code was directed towards:

1.   Obtaining the maximum amount of data for direct correlation with the experimental test data.

2.   Providing adequate comparisons between the various analytic solution modes.

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Note that the distinction is made here between a
solution based on the analytic blast model and a solution
using arbitrary (may be experimental) local overpressure
data.

The scope of the analysis is presented in Table 7 where
DEPROP and DEPROB denote, respectively, the panel and beam
solutions based on the analytic blast models. KADBOP and
KADBOB denote the panel and beam solutions, respectively,
using arbitrary overpressure data inputs.

Beam representation of a panel using the analytic
overpressure model is denoted by KADBOB-A.

RESULTS CORRELATION

This part of the report is devoted to the presentation
of the analytic results and their correlation with the
experimental data obtained from the test conducted on the
A-4C aircraft.

The analytic results obtained for each method are pre-
sented in separate paragraphs which include comprehensive
discussions. For each case, the discussion describes the
area and the degree of correlation between the analytic and
experimental data.

METHOD 1

The sure-safe and sure-kill envelopes constructed
according to the procedure for Method 1 are presented in
Figs. 38 and 39, respectively. For a ground detonation and
a parked aircraft, corresponding to the DICE THROW event
configuration, the sure-safe and sure-kill ranges shown on
these figures are 693 m (2275 ft) and 210 m (690 ft),
respectively.

Only a generalized and largely qualitative correlation
between these results and the experimental data is possible
since Method 1 is limited in scope to the determination of
the critical range values according to the specified sure-
safe and sure-kill criteria. These range values which are
based on predetermined allowable peak overpressures do not
offer a suitable basis for correlation with the experimental
data obtained for different overpressure conditions. However
restrictive this situation, revealing observations can be
made in assessing validity of this method.

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TABLE 7. Chart of the NOVA-2 Solution Runs.

Structural Element	Panel Representation		Beam Representation		
	DEPROP	KADBOP	DEPROB	KADBOB	KADBOB-A*
Panel 1	X	X	-	X	X
Panel 4	X	X	-	X	-
Panel 5	X	-	-	X	-
Panel 8	-	-	-	X	-
Longeron 5	-	-	X	X	-
Frame 277	-	-	X	-	-

*KADBOB-A denotes the solution of a panel represented as
a beam subjected to the analytic overpressure entered as
a point by point input.
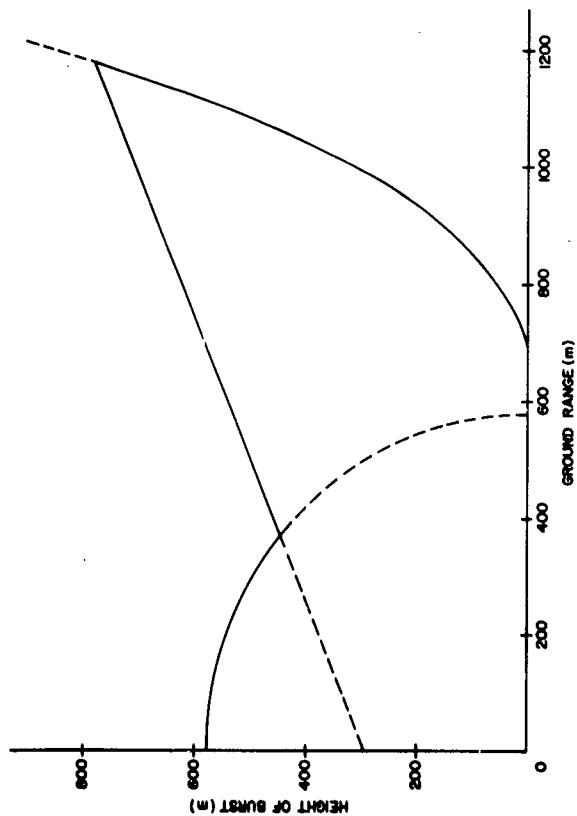
FIGURE 38. Sure-Safe Envelope, Method 1.

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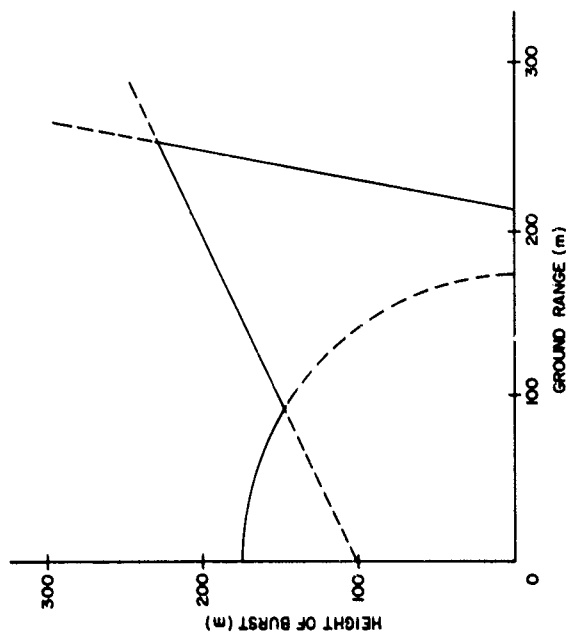
FIGURE 39. Sure-Kill Envelope, Method 1.

The assessment of damage sustained by the instrumented A-4C aircraft located at 375 m (1230 ft) from the GZ suggested that the sure-safe condition was exceeded at 41.4-kPa (6 psi) peak overpressure. This was indicated by localized damage particularly in the fuselage STA 277 region, Fig. 25, where the failure of the frame to lower longeron joint led to panel skin rupture.

The experimental results show that this joint was the critical structural element. Unfortunately, no strain records were obtained for that area because this was not previously predicted to be the weak element.

However, the localized nature of the failure implies that the loading was not grossly in excess of yield; other-wise, more widespread failure would have resulted. This, in turn, leads to the final observation that, for A-4C aircraft, the vulnerability prediction according to Method 1 for sure-safe condition is decidedly conservative.

The consideration of the sure-kill condition presents a directly opposite picture. Here, the analytic value of 210 m (690 ft) for the critical range corresponding to 137.9-kPa (20 psi) peak overpressure is well in excess of 62.1-kPa (9 psi) peak overpressure which was sufficient to produce the catastrophic failure of the aircraft located 311 m (1020 ft) from the GZ. This immediately suggests that the analysis is severely unconservative.

There is no accurate way to predict from the available experimental data the lowest overpressure level which would have produced a catastrophic failure. Assuming that 62.1 kPa (9 psi) represented a minimum overpressure value for a catastrophic failure to occur, this is equivalent to at least 120 percent error in the critical overpressure postu-lated by the analysis.

In the final assessment, the test has demonstrated that Method 1 applied to the aircraft in the fighter and fighter-bomber classes, as represented by A-4C aircraft, tends to be conservative in predicting the critical range for sure-safe condition, but unconservative in predicting the critical range for sure-kill condition. However, the extent of conservatism and unconservatism fall within the error limits specified in the method formulation.

The plots of the analytic solutions presented in Figs. 38 and 39 appear rather crude. The multiple ground range values obtained for a given height of burst are not realistic and tend to be confusing.

For an analyst familiar with Method 1, the analysis requires an estimated 20 minutes of engineering time to perform and less than one second computer time to execute on a large computer. The method provides a means for rapid determination of the vulnerability envelopes and can be satisfactorily used where a solution error factor of two is acceptable.

METHOD 2

Skin Panels

The analytic data obtained for three panels are presented in Table 8 and are applicable to the sure-safe condition only. They represent three sets, each consisting of ten critical overpressure values related to the angle of incidence, theta, which is defined as the angle subtended between the panel surface and the direction of shock propagation.

For panel 1, the critical overpressure of 27.2 kPa (3.94 psi) was indicated at 90-degree angle of incidence which corresponded to the actual panel orientation during the test.

With incidence angles of approximately 30 and 90 degrees for panels 5 and 6, respectively, the corresponding critical overpressure values of 46.9 kPa (6.8 psi) and 31.2 kPa (4.52 psi) were obtained.

The selection of panels 1, 5, and 6 for the skin panel representation in the analysis was made to provide the best possible correlation between the analytic and experimental data. These panels were suitably instrumented with strain gages and produced satisfactory strain data during the test.

However, the overall survey of the strain data indicated that the actual panel response falls somewhere between the idealized simply supported and clamped edge boundary conditions. This was particularly apparent for panels 1, 2, and 4. For these panels, strains of comparable magnitudes were obtained at the panel center and at mid-points along the longer edge.

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TABLE 8. Peak Experimental Strain Data Summary
for Panels 1 through 8.

Panel No.	Panel Center Inside Surface		Panel Longer Edge Outside Surface	
	Microstrains	Gage No.	Microstrains	Gage No.
1	3400	1	2600 2880	9 10
2	-	-	2200 2180	15 17
3	3900	19	-	-
4	5000	24	6000	27
5	-	-	8000 8800	31 32
6	3200	34	7600	35
7	-	-	4600	37
8	3800	40	2400	42

NOTE: (a) Strain gage locations are shown in Figs. 8
through 15.
(b) Peak strains are tensile in every case.

It appears, therefore, that the assumption of clamped edge boundary conditions for the panels does not necessarily hold true in the actual true-to-life aircraft structure. The less than rigid quality of the panel boundary substructure comprising panel boundaries and the structural deterioration of the entire aircraft structure through the years of service should be considered among the factors likely to contribute to the departure from the idealized panel behavior.

The apparent inconsistency between the experimental data and the behavior of idealized panels with clamped edge boundary conditions can also be attributed to the fact that the exact locations of panel boundaries could not be determined. The location of the panel boundary depends on the configuration of substructure and its rigidity. Where the substructure is less than rigid, the effective boundary location also depends on the deformation which is a function of the loading. Consequently, the strain gages installed in the vicinity of the panel edge, where steep strain gradients prevailed, produced data which could be easily misinterpreted.

There are two alternatives. On one hand, there is a possibility that a skin panel, which constitutes an element of a larger structural component, does not necessarily respond according to idealized, clamped edge boundary conditions. On the other hand, the experimental data obtained from even slightly misplaced strain gages can easily lead to erroneous observations.

At this stage, no sufficient evidence is available to ascertain which of these possibilities should play the dominant role in the experimental data interpretation. This situation severely complicates any attempt to verify Method 2 on the basis of the experimental data.

However, closer examination of the strain gage data summarized in Table 8, in conjunction with Figs. 8 through 15, further implied that the actual panel response was somewhere in between the clamped edge and simply supported boundary conditions. Steep strain gradients in the vicinity of the panel edge did exist, but strains of comparable magnitude might have also been present at the center of the panel. The typical response for a panel with clamped edge boundary condition was not entirely evident in the experimental data for panels 5 and 6. The thinner skin of panel 5, as compared to panel 1, naturally developed higher strains (gages 31 and 32 as compared to gages 9 and 10). These high

strains might have been accompanied by comparably high
strains at the panel center in a pattern similar to that of
panels 1, 2, and 4. The local failure of panel 6 substruc-
ture no doubt affected the skin strains to the extent that
the panel response cannot be represented by the clamped edge
condition.

From the foregoing arguments emerges a postulate that
the experimental data reflected with a fair degree of
accuracy the actual strains developed in skin panels during
the test. Based on that postulate, qualifying statements
regarding Method 2 can now be made.

The sure-safe condition for Method 2 implies impending
yield of material. For the biaxial stress field this can be
defined by

$$F_y = \frac{E \, \epsilon_y}{1 - \nu^2}$$

where F_y (63000 psi) is the yield stress,
E^y (10.4 x 10^6 psi) denotes the Modulus of Elasticity,
and ν (0.33) in the Poisson's Ratio.

Then, the corresponding yield strain, ϵ_y, in the
direction normal to the panel longer edge is approximately
5400 microstrains.

The yield condition for panel 1 indicated by the
analysis was at 27.2-kPa (3.94 psi) peak overpressure.
However, the experimental data showed no yield even at
41.4-kPa (6 psi) peak overpressure. In fact, only 60 per-
cent of yield was reached.

A directly opposite picture is presented in the case of
panel 5, for which the yield condition was obtained analyti-
cally as 46.9-kPa (6.8 psi) peak overpressure. According to
the experimental data, at 41.4-kPa (6 psi) peak overpressure,
the yield condition was already exceeded by 48 percent of
the yield strain value.

Considerable discrepancies between the analytic and
experimental data are in evidence for the panel structural
elements. Within these discrepancies, the analytic values
of critical peak overpressure may be higher or lower than
the corresponding experimental data by an appreciable margin.
However, since the degree of accuracy of the experimental
data could not be adequately determined, there is insuffi-
cient evidence to verify or deny the validity of Method 2.

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In any case, the observed discrepancies appear to fall within the error factor of 1.6 specified by the method for the vulnerability ranges determined in the sure-safe condition analysis.

Method 2 is distinctly superior to Method 1 as it provides for a selection and analysis of weakest structural elements which are individually modeled in some detail. The analytic solutions using OVPR2 code are readily obtained in less than 10 seconds of computer time on a large computer.

Longerons

For the sure-safe condition, the analysis modeled longeron 5. The analytic results are presented in Table 4. Shock incidence at that location was 32 degrees and the critical peak overpressure was assessed at 65.6 kPa (9.51 psi).

For the sure-kill condition, the lower longeron was used in the analysis and the resulting critical peak over-pressure values are summarized in Table 5. For the actual lower longeron location, the shock incidence angle was approximately 90 degrees and the critical peak overpressure was 50.3 kPa (7.3 psi).

The most striking result of the longeron analysis is that a higher critical peak overpressure was indicated for the sure-safe condition than for the sure-kill condition. This apparently irrational result was not altogether unexpected when one considers the modeling process for longeron elements prescribed by the method.

The weakest longerons selected for the sure-safe condition analysis were closely spaced. Consequently, the effective pressure loads sustained by an individual longeron were relatively small and a large critical peak overpressure value was predicted by the analysis. For the sure-kill condition, the strongest longerons were modeled; however, they were spaced far apart. This was equivalent to a disproportionately higher pressure load being sustained by this longeron and produced a lower critical peak overpressure value.

It appears, therefore, that very careful consideration, based primarily on the analytic experience, must be exercised in the selection of longeron elements to assure compatible analytic results.

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In the case of A-4 aircraft, the selection of longeron elements for modeling was difficult. The fuselage structure was made up of closely spaced frames and ring-like, stiffener members. Typical stringers were absent and the longitudinal structural members were limited to two types of longerons represented in the analysis.

Adequate strain gage instrumentation of the longerons was not available for the test because of limited recording channels, and other sources of relevant experimental data were totally absent. However, some insight into the overall structural integrity was gained from the extent of damage sustained by the aircraft exposed to different overpressure levels. At 41.4-kPa (6 psi) free-field peak overpressure, no damage of longerons was evident. At 62.1-kPa (9 psi) overpressure, the catastrophic failure of the fuselage, initiated in the vicinity of the frame STA 277, progressed through the weak longerons and led to a rupture of the strong longerons and the entire section of the fuselage.

For the specified test conditions, the longeron vulnerability between no damage and catastrophic failure are equivalent to a 64 m (210 ft) change in the range from the GZ, i.e., from 375 m (1230 ft) to 311 m (1020 ft).

The critical free-field peak overpressures predicted according to Method 2 are equivalent to the ranges between 305 m (1000 ft) and 335 m (1100 ft) from the GZ. This is well within the 1.6 error factor value specified for this method. Therefore, the analytic results obtained for longerons are reasonable.

Fuselage Frame

The frame analysis data based on fuselage frame STA 277 are presented in Table 6, and the indicated minimum critical peak overpressure value was 21.9 kPa (3.18 psi). According to the free-field overpressure data collected during the test, this pressure level was recorded approximately 610 m (2000 ft) from the GZ.

Considering that a damage of fuselage frame STA 277 was sustained by both aircraft fielded 375 m (1230 ft) from the GZ, the analytic prediction may be assessed outright as at least reasonable. It almost falls within the 1.6 error factor value specified for the method. However, noting the extent of damage, it is immediately suggested that yielding

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of the material had to begin at substantially lower load
level. This can be readily deduced by examining Fig. 25
which shows failure of the joint between frame STA 277 and
the lower longeron in the instrumented aircraft BUNO 149588.
There, sheared fasteners and the displacement of the attach-
ing angle from its normal location are clearly visible. The
displacements of that magnitude were associated with con-
siderable plastic deformation preceded, of course, by yield-
ing. In the second aircraft BUNO 145062, failure of frame
STA 277 was more severe and more extensive. Besides the
joint failure, the frame flanges buckled and ruptured higher
up the span, approximately 0.61 m (2 ft) above the lower
longeron joint, Fig. 30. It was interesting to note that
the flange failure occurred at a cutout hole in the frame
web, which was present in aircraft BUNO 145062 but absent in
aircraft BUNO 149588.

     With this demonstration that the frame yielded at an
overpressure level distinctly below 41.4 kPa (6 psi) and
therefore closer to the predicted value of 21.9 kPa (3.18 psi),
experimental verification of Method 2 for the frame element
analysis is <u>satisfactorily accomplished</u>.

METHOD 3

     Results of the analysis, covering all options available
in NOVA 2, are presented in a series of plots for strain
data versus time, with zero-time corresponding to the instant
of blast intercept. Wherever applicable, plots of pertinent
experimental data are also included, superimposed on the
analytic data.

     Five skin panels, one longeron and one frame represent
the three types of structural elements analyzed using the
NOVA 2 code. Each element is covered in separate paragraphs
which follow.

Skin Panels

     The overriding objective of this study was to implement
a correlation between the experimental and analytic data.
Thus, considerable effort was initially made to provide a
representation of various panel configuration parameters in
order to assess their effect on panel response to blast.
However, as mentioned earlier, restrictions imposed by
practical considerations reduced the parametric aspect of
this study to:

1.   Flat and curved panels.

2.   Clamped and simply supported edge boundary conditions.

3.   Panels of different width.

A comparison between a flat and a curved panel was provided by panels 1 and 5 respectively. Panels 1 and 8 represented the clamped and simply supported edge boundary conditions, respectively. Panels 4 and 5 illustrated the effect of the width parameter variation on the panel response. Panel 6, which was also covered analytically, should be considered separately. Its slightly trapezoidal rather than rectangular shape renders it unsuitable for a direct correlation with other panels.

The data plots for these five panels are presented in three sets. Figures 40 through 49 represent strain and deflection data obtained from the solutions based on the analytic blast model.

The panel solutions based on the experimental over-pressure data (KADBOP solutions) are illustrated by Figs. 50 through 54. The corresponding data for beam representation of panels (KADBOB solutions) are given in Figs. 55 through 67.

The analytic blast model and the experimental over-pressure data are correlated in Figs. 68 through 72 for each panel analyzed.

The panel response observed during the test has been described in general terms in the assessment of Method 2. Additional and more detailed observations will follow as more pertinently applicable to the assessment of Method 3.

Several comments concerning the overpressure charac-teristics related to structural elements in general seem appropriate at this point. Examination of the pressure gage records revealed considerable variation of overpressure across structural elements, such as skin panels, during the blast diffraction phase. (See pressure gage data in Volume II. of NWEF Report 1145, Ref. 6.) The time of shock arrival was found generally to vary from point to point on the panel
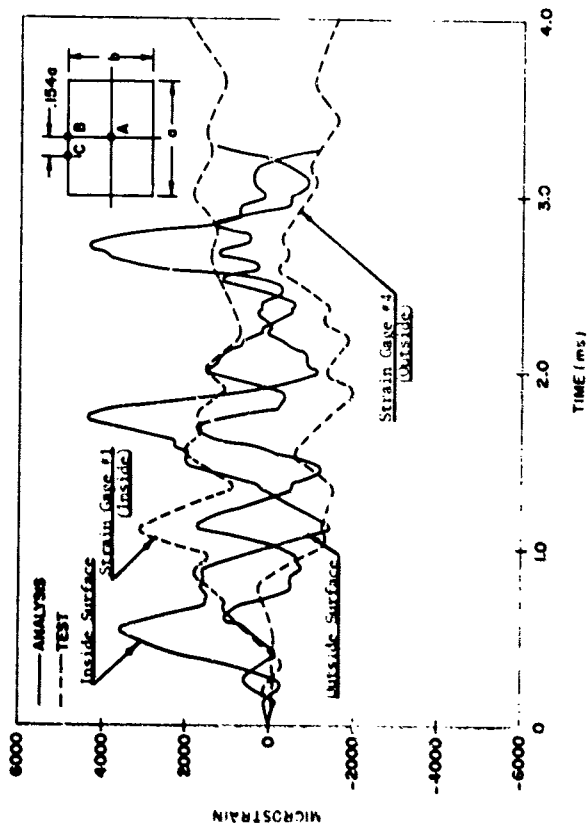
FIGURE 40. Strain Data Correlation for Panel 1 - Location A.
NOVA 2 - DEPROP Solution.
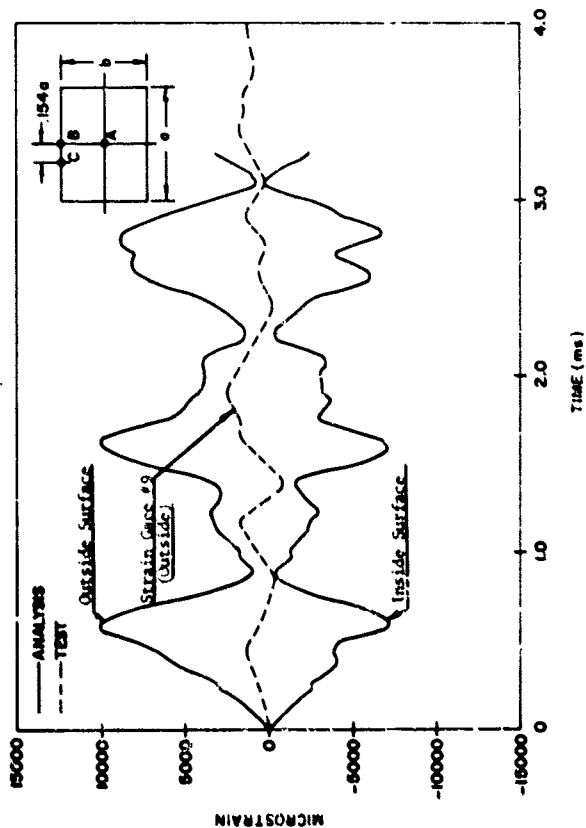
FIGURE 41. Strain Data Correlation for Panel 1 - Location B
NOVA 2 - DEPROP Solution.
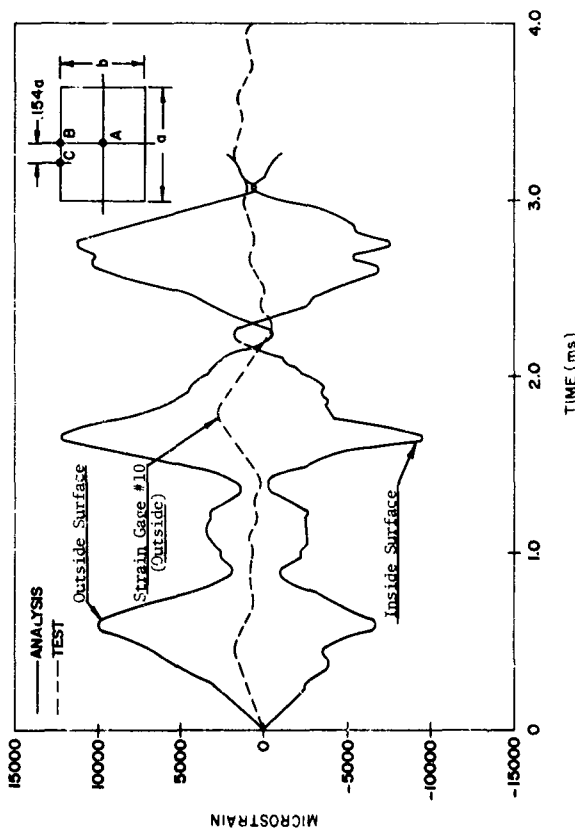
FIGURE 42. Strain Data Correlation for Panel 1 - Location C
NOVA 2 - DEPROP Solution.

88
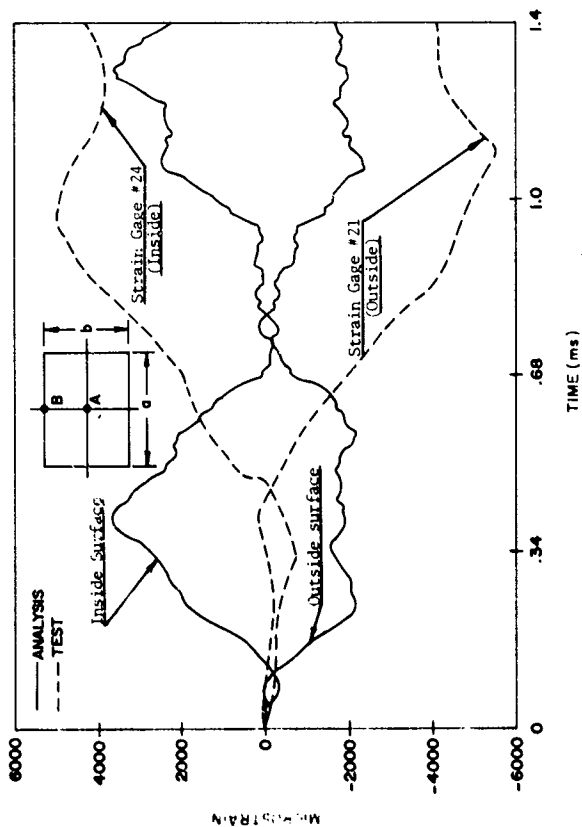
FIGURE 45. Strain Data Correlation for Panel 4 - Location A
NOVA 2 - DEPROP Solution.
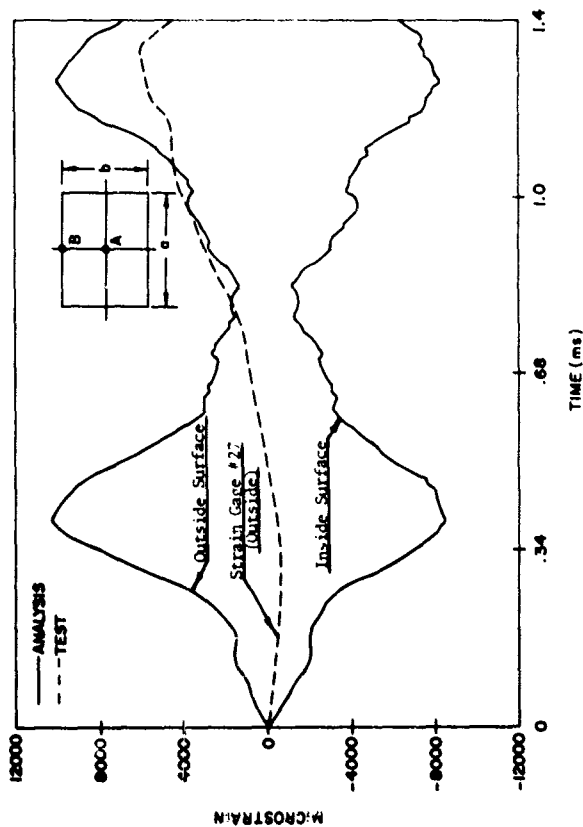
FIGURE 44. Strain Data Correlation for Panel 4 - Location B
NOVA 2 - DEPROP Solution.
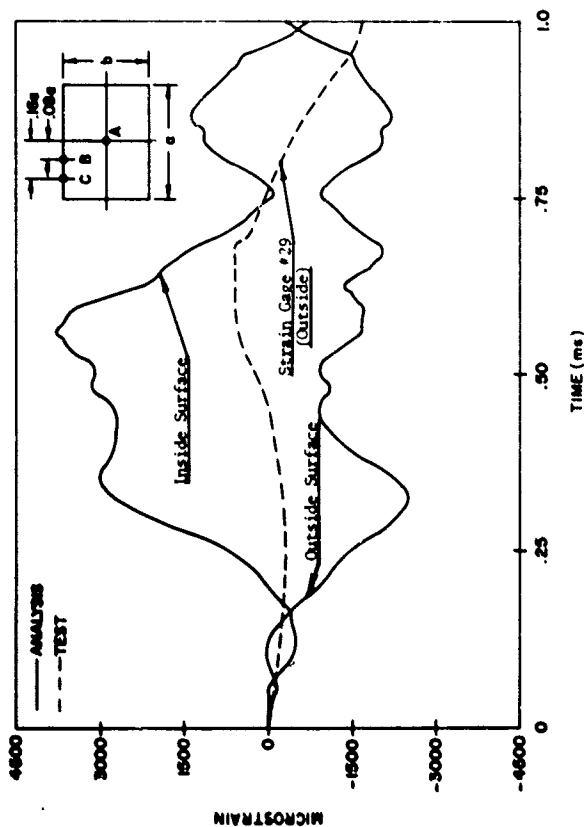
FIGURE 45. Strain Data Correlation for Panel 5 - Location A NOVA 2 - DEPROP Solution.

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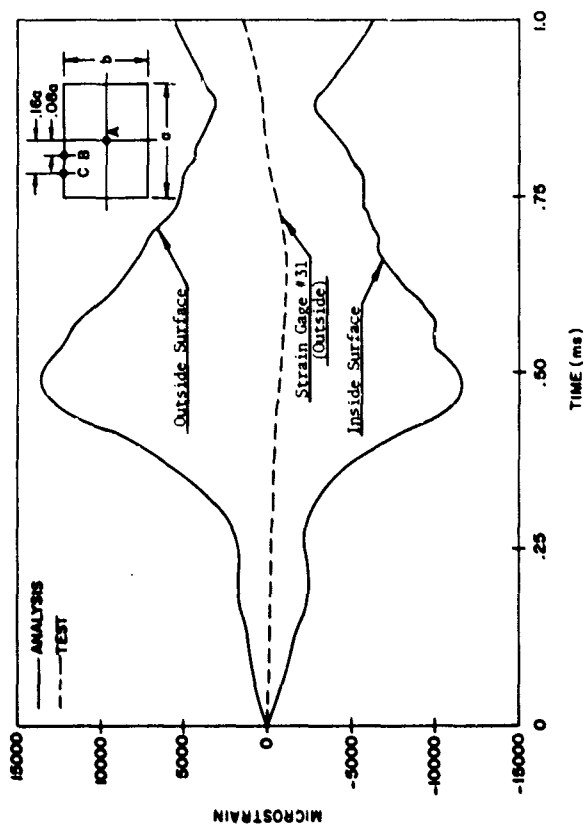
FIGURE 46. Strain Data Correlation for Panel 5 - Location B
NOVA 2 - DEPORP Solution.

FIGURE 47. Strain Data Correlation for Panel 5 - Location C
NOVA 2 - DEPROP Solution.

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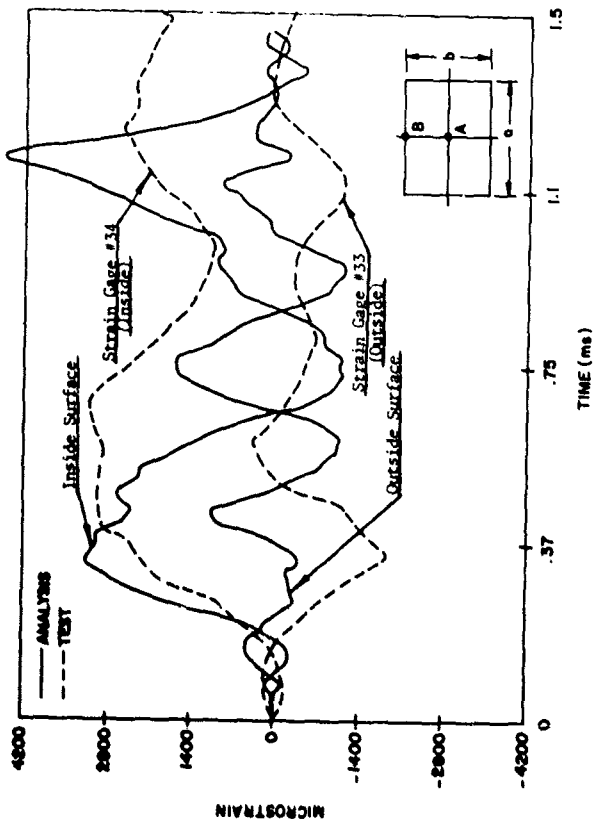
FIGURE 48. Strain Data Correlation for Panel 6 - Location A NOVA 2 - DEPROP Solution.
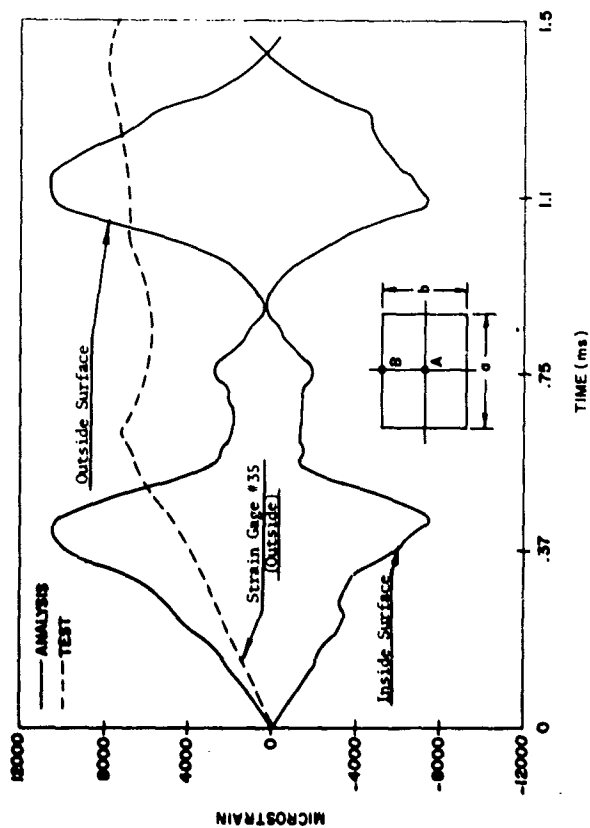
FIGURE 49. Strain Data Correlation for Panel 6 - Location B
NOVA 2 - DEPROP Solution.
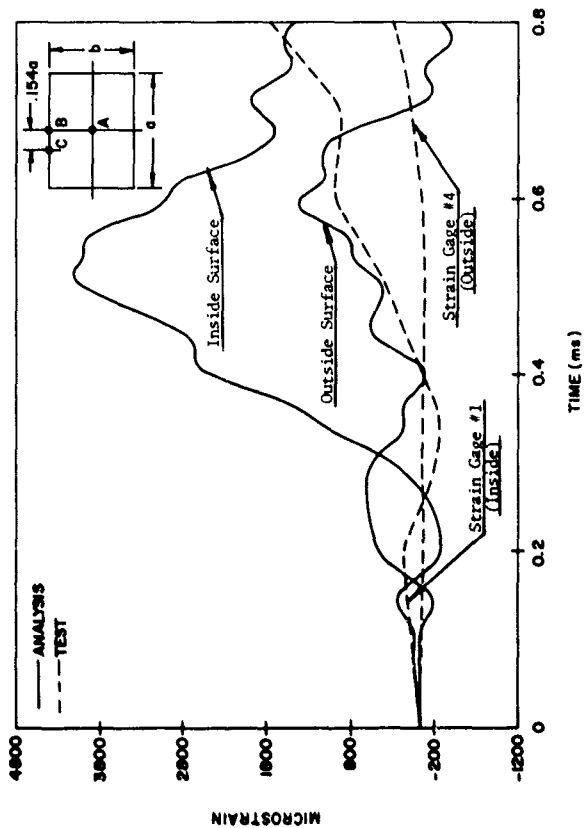
FIGURE 50. Strain Data Correlation for Panel 1 - Location A
NOVA 2 - KADBOP Solution.

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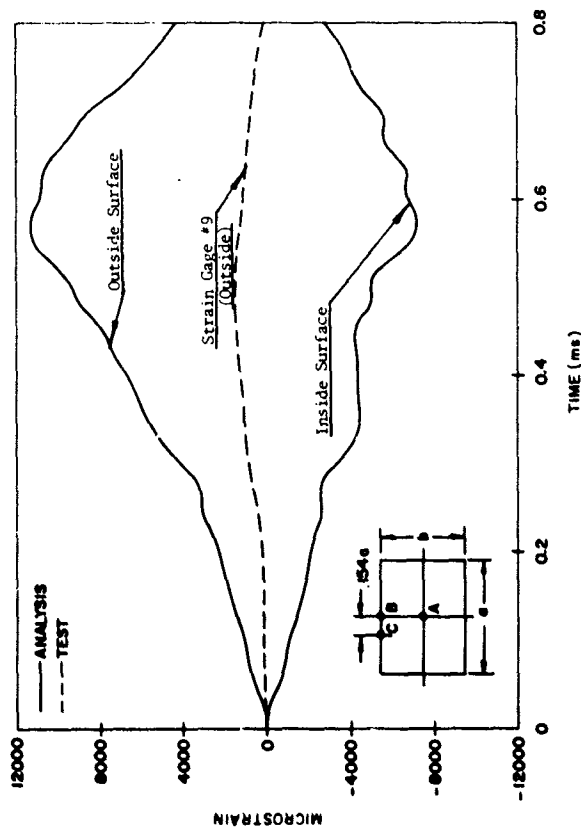
FIGURE 51. Strain Data Correlation for Panel 1 - Location B
NOVA 2 - KADBOP Solution.
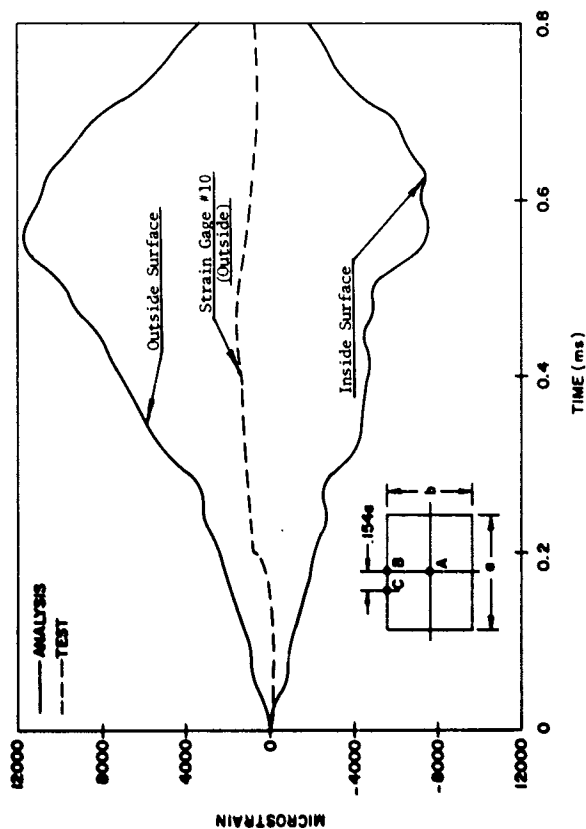
FIGURE 52. Strain Data Correlation for Panel 1 - Location C
NOVA 2 - KADBOP Solution.
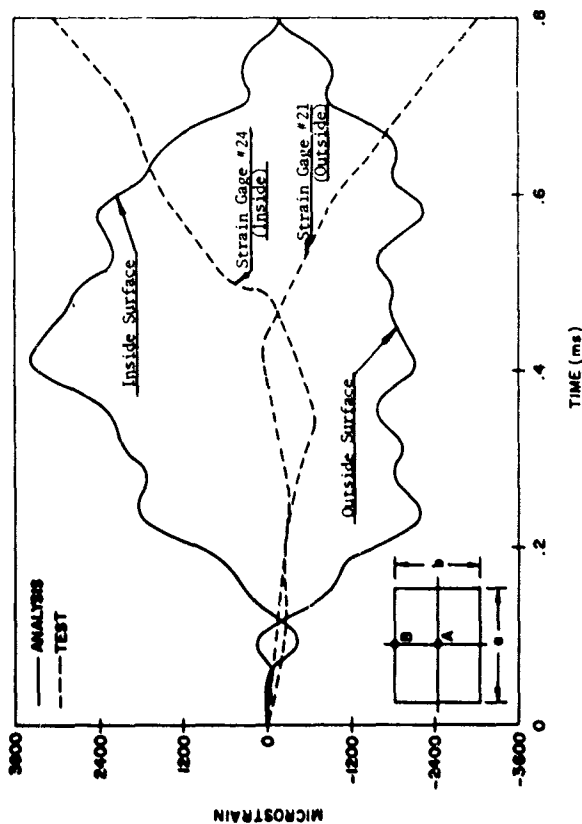
FIGURE 53. Strain Data Correlation for Panel 4 - Location A NOVA 2 - KADBOP Solution.

99
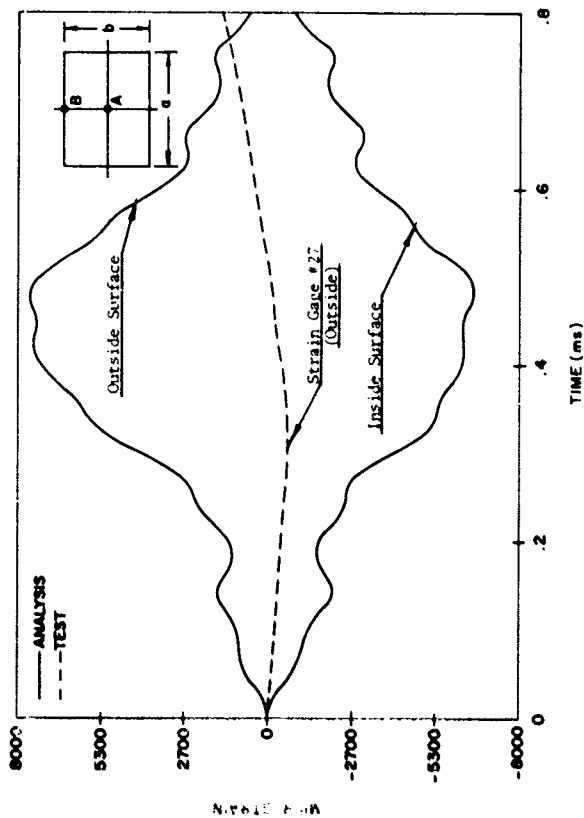
FIGURE 54. Strain Data Correlation for Panel 4 - Location B
NOVA 2 - KADBOP Solution.
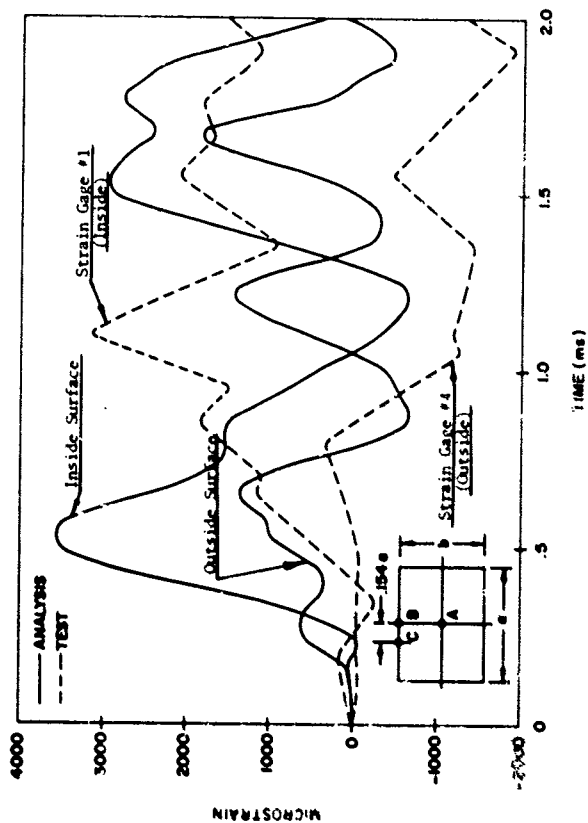
FIGURE 55. Strain Data Correlation for Panel 1 - Location A
NOVA 2 - KADBOB-A Solution (Ref. TABLE 7).

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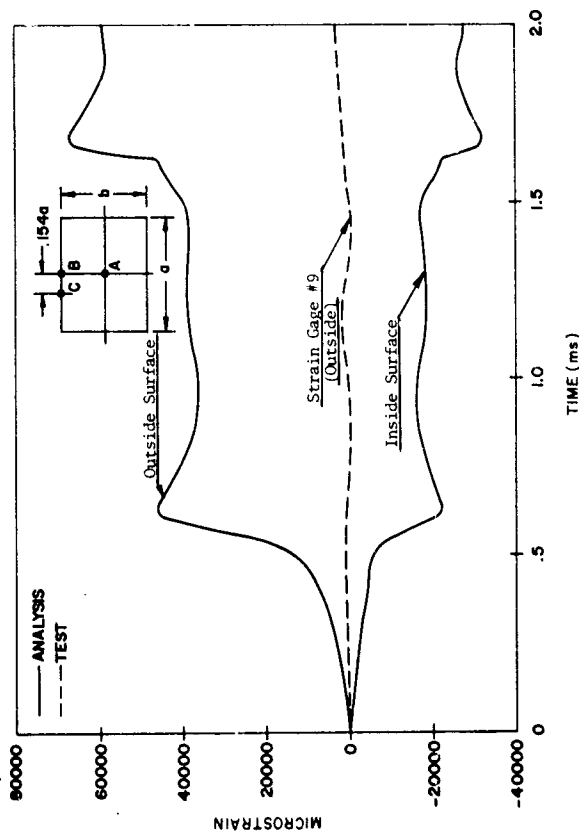
FIGURE 56. Strain Data Correlation for Panel 1 - Location B
NOVA 2 - KADBOB-A Solution (Ref. TABLE 7).
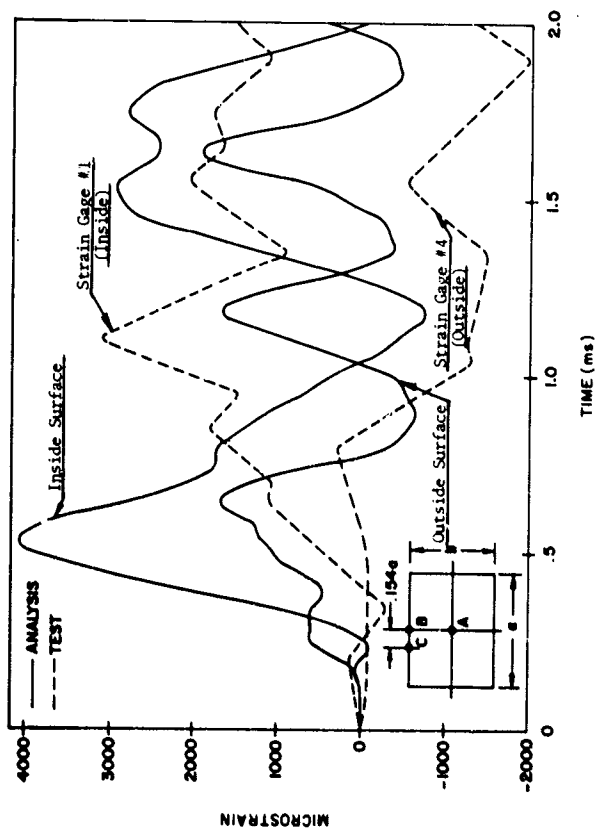
FIGURE 57. Strain Data Correlation for Panel 1 - Location A
NOVA 2 - KADBOB Solution.

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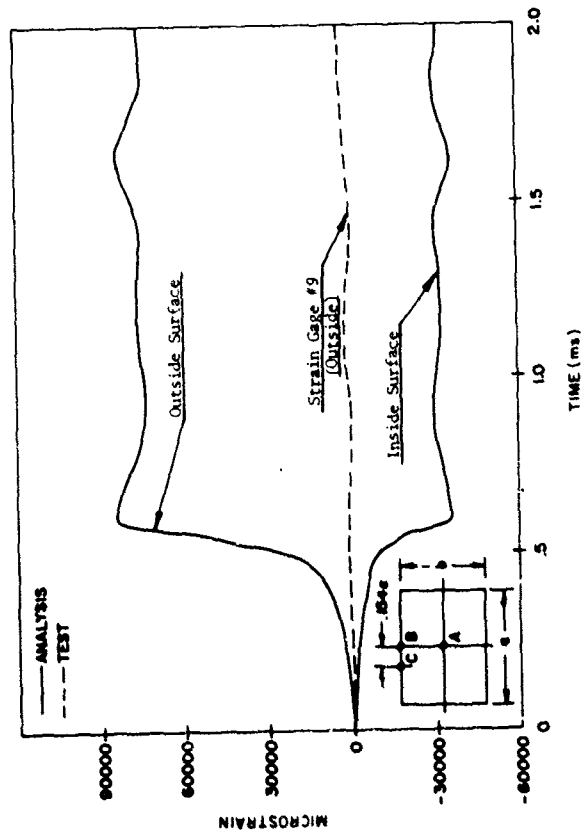
FIGURE 58. Strain Data Correlation for Panel 1 - Location R NOVA 2 - FADBOB Solution.
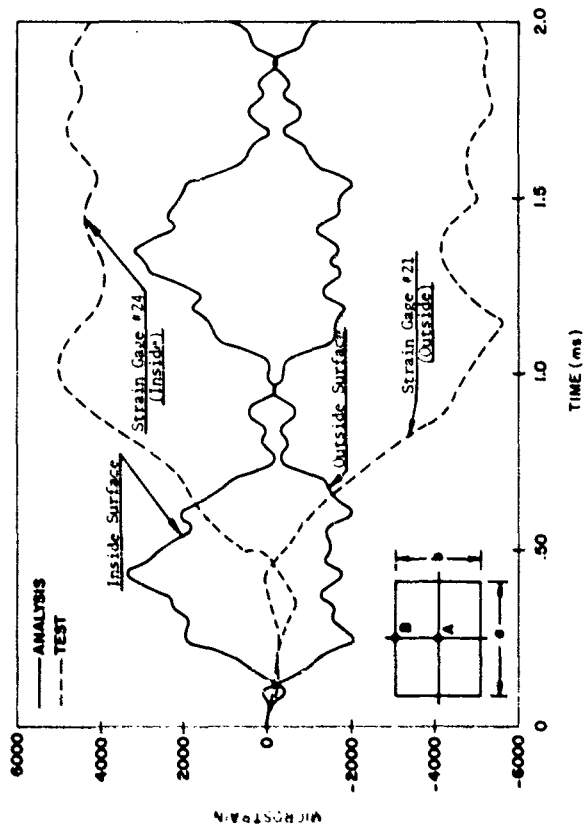
FIGURE 59. Strain Data Correlation for Panel 4 - Location A
NOVA 2 - KADBOB Solution.
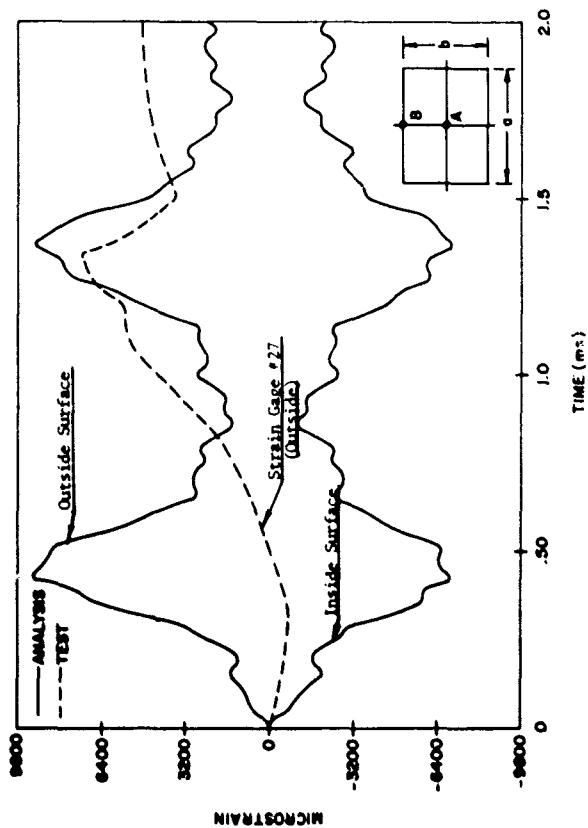
FIGURE 60. Strain Data Correlation for Panel 4 - Location B
NOVA 2 - KADBOB Solution.
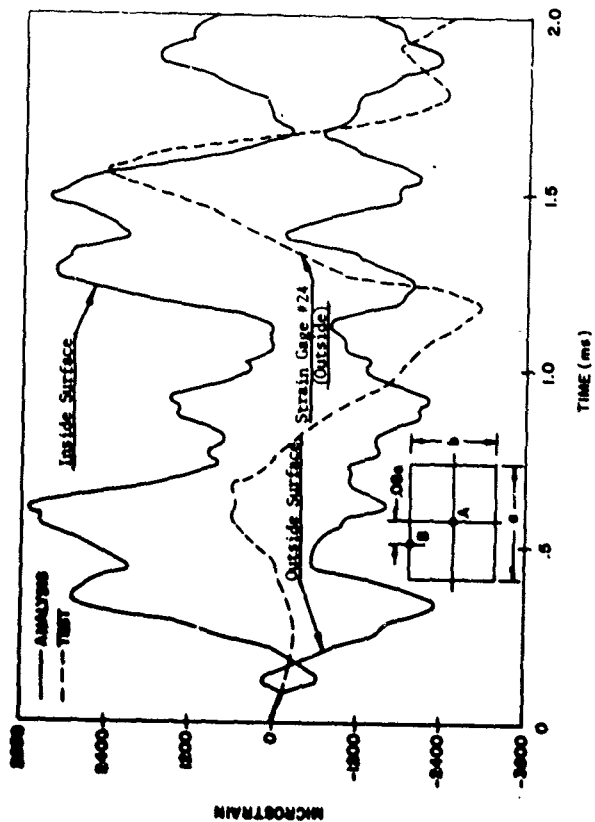
FIGURE 61. Strain Data Correlation for Panel 5 - Location A
NOVA 2 - KADBOB Solution.
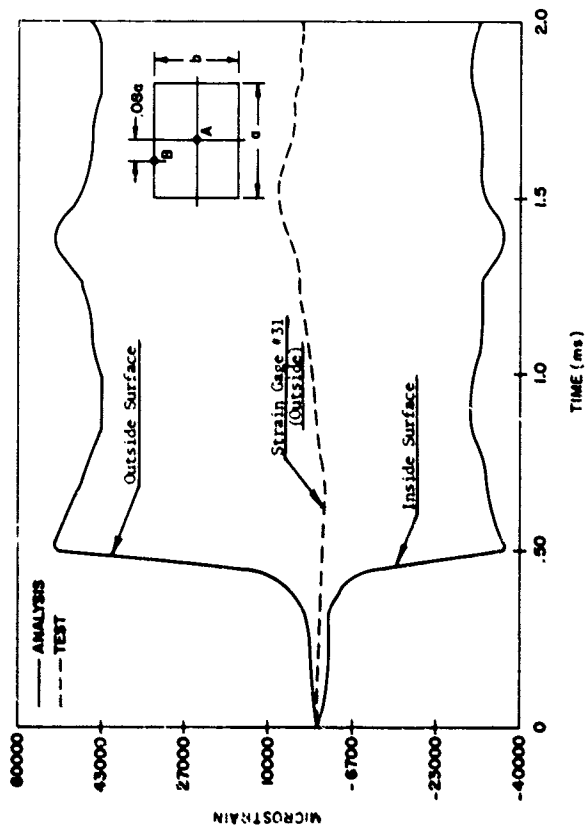
FIGURE 62. Strain Data Correlation for Panel 5 - Location B
NOVA 2 - KADBOB Solution.
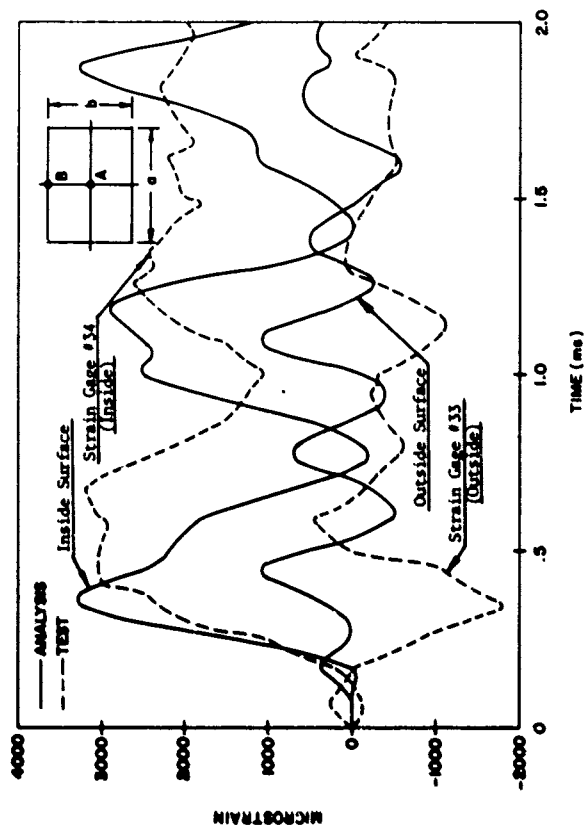
FIGURE 65. Strain Data Correlation for Panel 6 - Location A
NOVA 2 - KADBOB Solution.
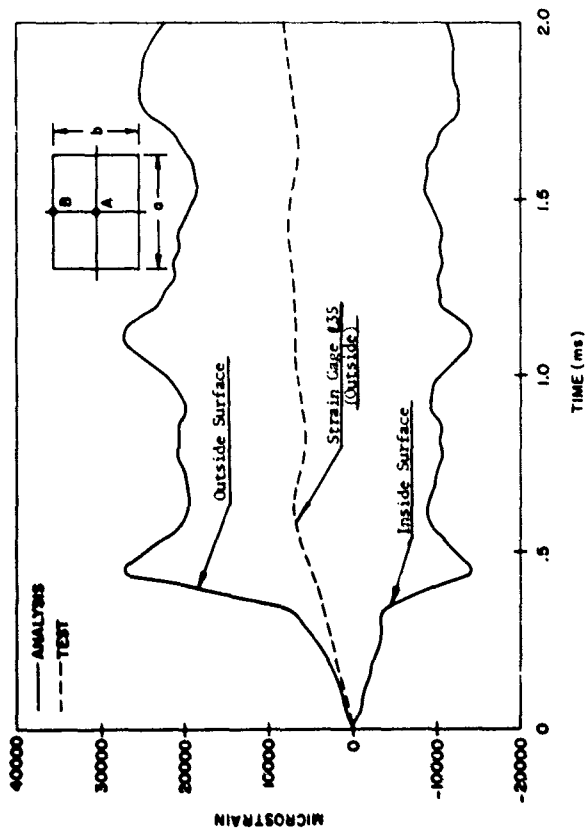
FIGURE 64. Strain Data Correlation for Panel 6 - Location B
NOVA 2 - KADBOB Solution.

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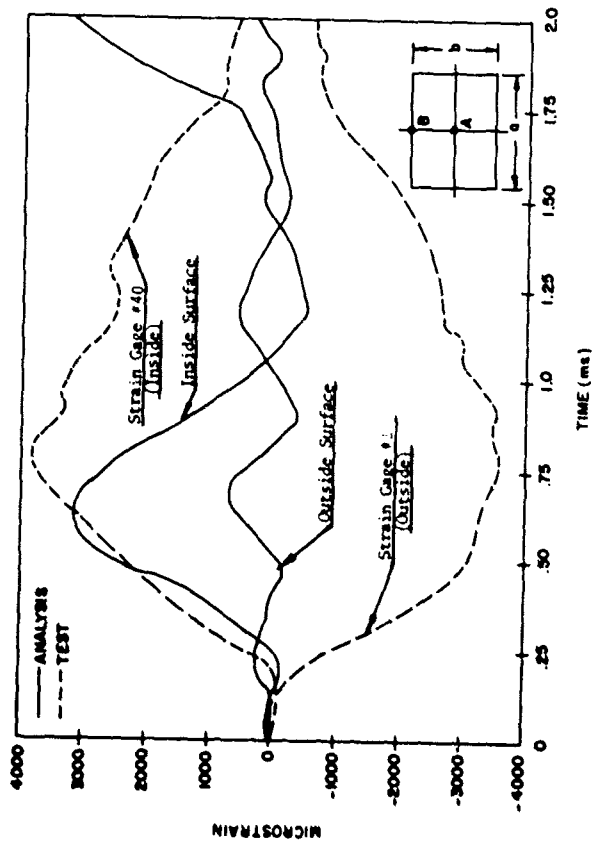
FIGURE 65. Strain Data Correlation for Panel 8 - Location A
NOVA 2 - KADBOB Solution - Simply Supported Edges.
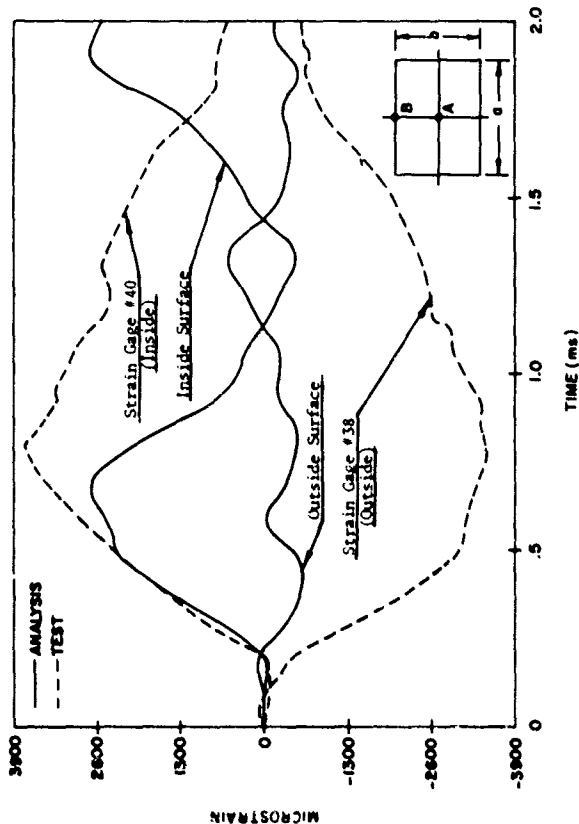
111

FIGURE 66. Strain Data Correlation for Panel 8 - Location A
NOVA 2 - KADBOB Solution - Clamped Edges.
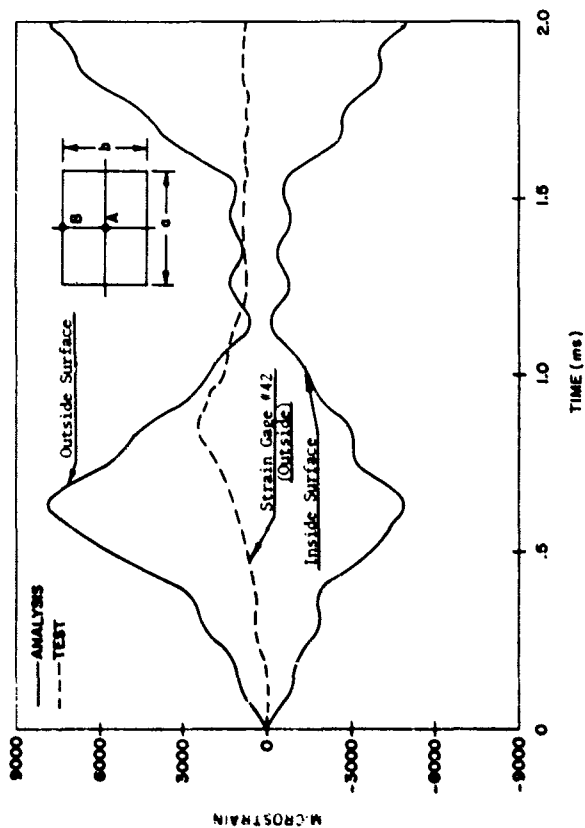
FIGURE 67. Strain Data Correlation for Panel 8 - Location B
NOVA 2 - KADBOB Solution - Clamped Edges.
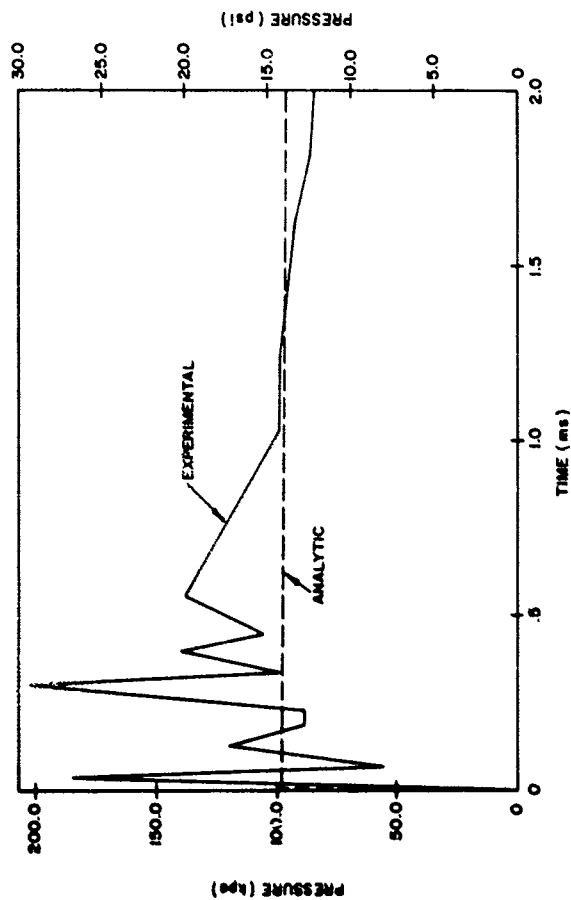
FIGURE 68. Analytic and Experimental Overpressure
Profiles for Panel 1.

114
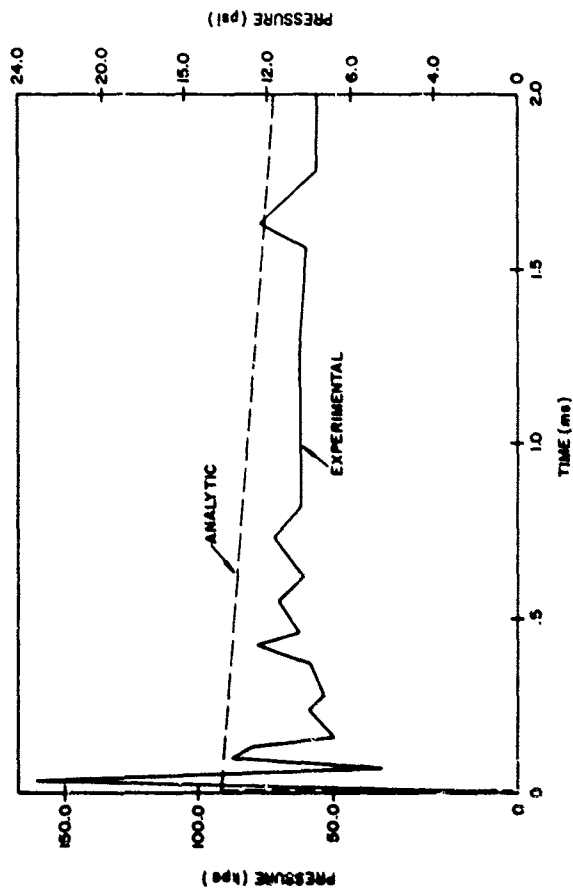
FIGURE 69.   Analytic and Experimental Overpressure
Profiles for Panel 4.

115
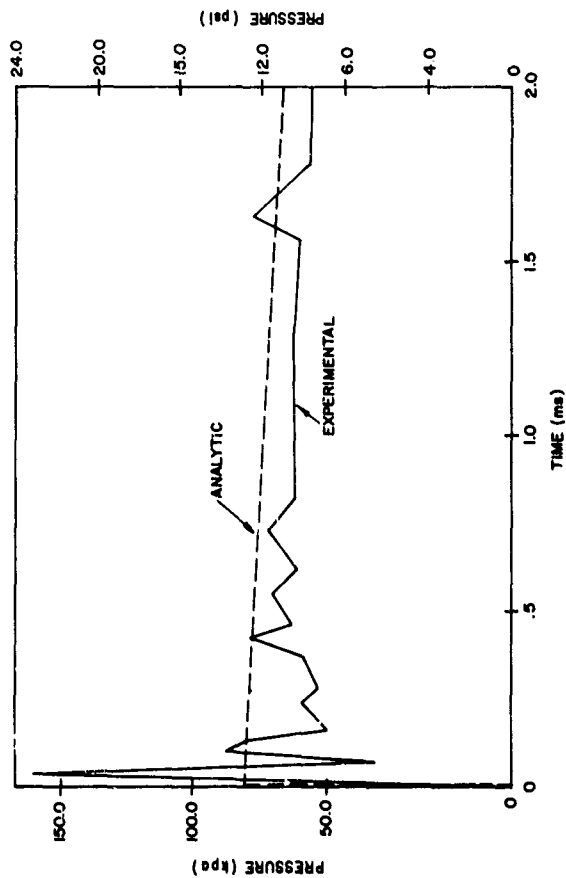
FIGURE 70. Analytic and Experimental Overpressure Profiles for Panel 5.

116
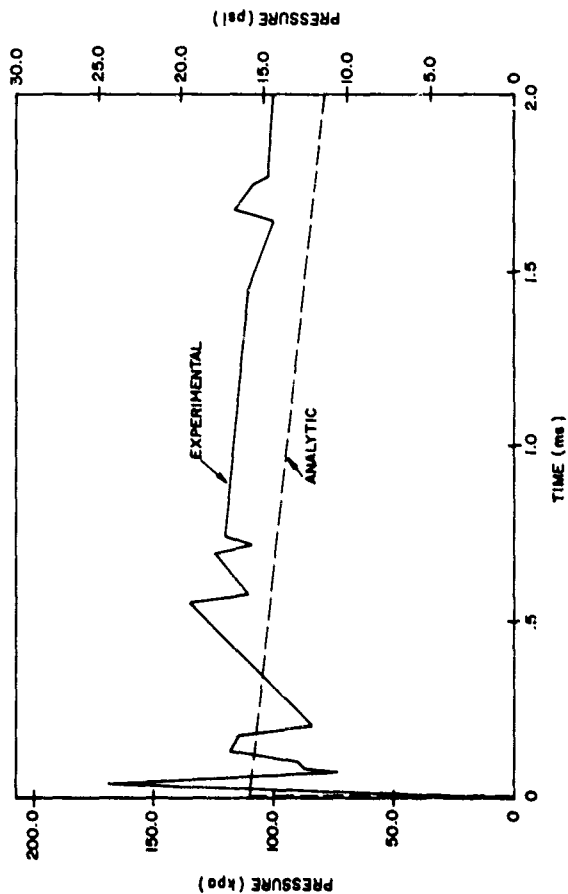
FIGURE 71. Analytic and Experimental Overpressure Profiles for Panel 6.

117
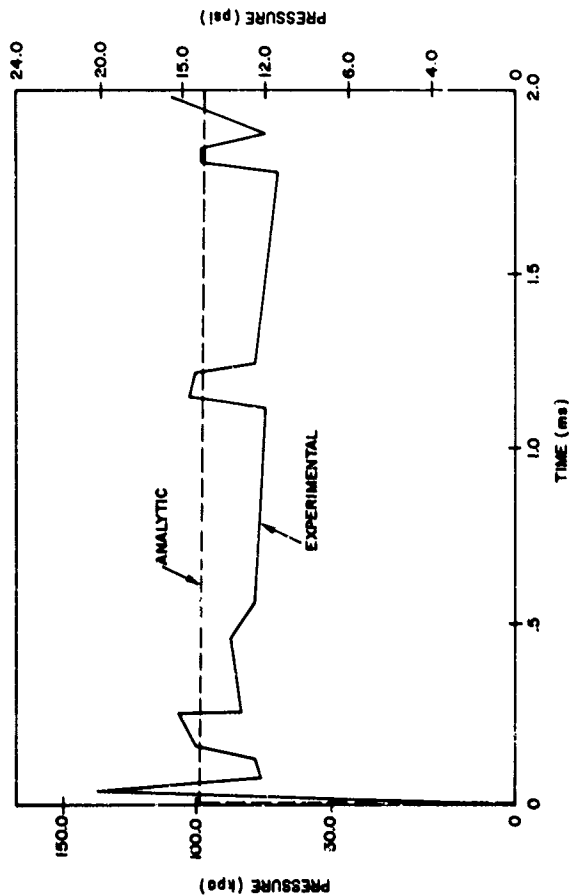
FIGURE 72. Analytic and Experimental Overpressure Profiles for Panel 8.

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surface. This was dependent on the panel shape and its orientation to the direction of blast propagation.

As mentioned earlier, the panel response did not conform either to the classic clamped edge or simply supported edge boundary conditions. Rather, an intermediate response was generally evident from the experimental data which is summarized in Table 8. This gives peak strains recorded at the center of the panels and along the longer edge near the panel boundary.

At the center of panel 1, for instance, the strain was 3400 microstrains, as compared to 2880 microstrains at the panel boundary. This implied a response more characteristic for panels with simply supported rather than clamped edge boundary conditions. A similar response could be observed for all panels located in the vertical stabilizer area. In contrast, the response of panels located on the fuselage appeared to approach more closely that of clamped edge boundary conditions. There, strains along the longer side near the boundary are higher than at the center of the panel.

A comparison of the peak strain values suggests that, generally, the response of the panels located in the vertical stabilizer area resemble more closely that of panels with simply supported edge boundary condition. The fuselage panels, however, approximate more closely responses for the clamped edge boundary conditions. This behavior appears to be related to the rigidity of the substructure at the panel boundaries. The substructure of the vertical stabilizer was more flexible than the fuselage substructure.

The survey of the strain gage data for the strain gradients developed near the panel boundary along the longer edges provided further evidence that the response of the panels during the test was between those responses typically expected for panels with simply supported boundary conditions and those with clamped edge ones.

For panel 1, a strain gradient of only 59 microstrains/ mm (1500 microstrains/in.) was indicated from the records of strain gages 5 and 9, Figs. 73 and 74. Much higher strain gradients were developed in panels 4 and 5. These were 640 and 970 microstrairs/mm (16,200 and 24,700 microstrains/ in.) respectively, as obtained from the data of strain

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FIGURE 73. Strain Gage No. 5 Data - Panel 1.

120
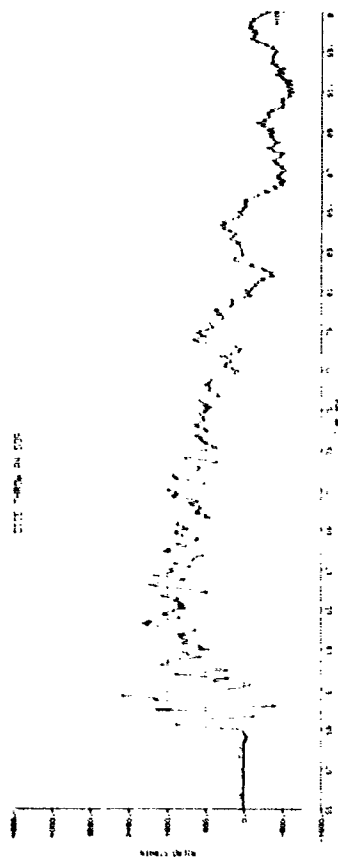
FIGURE 74. Strain Gage No. 9 Data - Panel 1.

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gages 27 and 28 for panel 4 and gages 31 and 43 for panel 5 (Figs. 75 through 78).*

An interesting pattern in tensile strain histories was observed for the flat and curved panels. At the center of flat panels, a maximum strain level was generally reached during the first oscillation. This was followed by the strain attenuation during subsequent cycles. At the panel edge, this was contrasted by several cycles of increasing strain amplitudes with the peak value reached, generally, on the third cycle. This initial amplification trend is followed by strain attenuation. Illustrations of these strain histories are presented in Figs. 79 and 74 for the center and the edge of panel 1, respectively. For the curved panels, the peak strain values both at the center and at the edge of the panel invariably occurred on the first oscillation, Figs. 80 and 75.

The study of the effect of the width parameter on panel response was limited to a comparison of strain data for panels 4 and 5. These panels, located side by side on the fuselage had a curvature of approximately 782 m (30.8 in.) radius. Both had identical dimensions except the width which was 73 and 87 mm (2.86 and 3.42 in.), respectively, for panels 4 and 5. Both panels responded in a similar manner and, as expected, the peak strain in panel 5, the wider panel, was substantially higher (8000 microstrains) than in the narrower panel 4 (6000 microstrains), as shown in Table 8.

In accordance with the fairly high natural frequency of the instrumented panels, their maximum response usually occurred within one millisecond after the blast intercept. A departure from this behavior was reflected in the strain histories obtained for some fuselage panels. It consisted of both high and low frequency oscillations. The high frequency oscillation reached peak strain during the first cycle following the blast intercept, but, typically, an even higher peak was registered at a later time in conjunction with the low frequency oscillation. This mode of response was most clearly apparent from the record of gage 22, on panel 4, Fig. 81, and shows the overall maximum response

*NOTE: Typically, a strain gage covers a finite area and its record represents average strain over that area. The data are, therefore, approximate values of the actual strain gradients that were induced during the test.

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FIGURE 75. Strain Gage No. 27 Data - Panel 4.
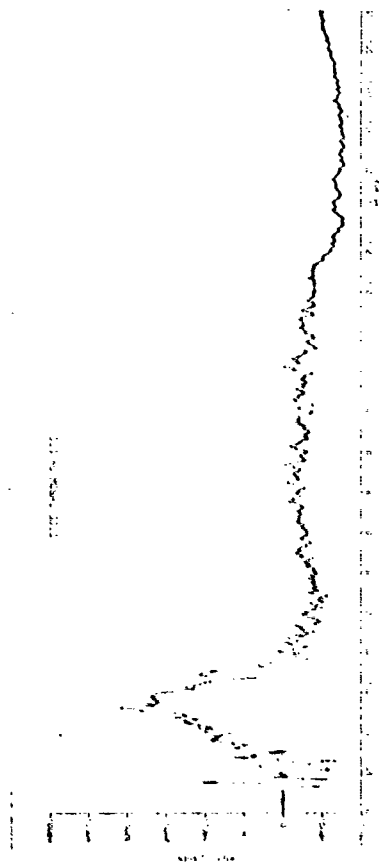
FIGURE 76. Strain Gage No. 28 Data - Panel 4.

124
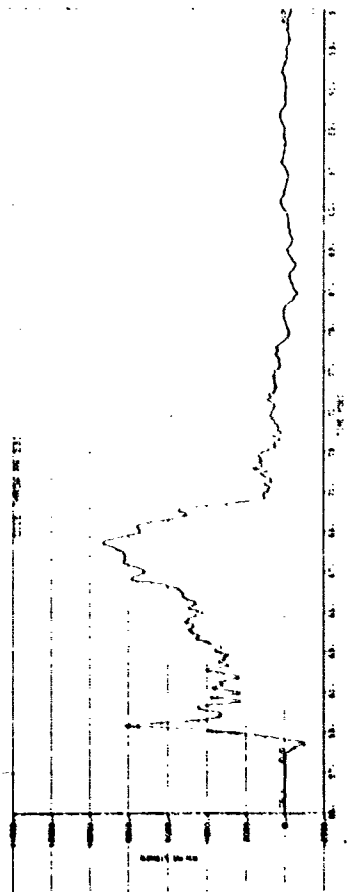
FIGURE 77. Strain Gage No. 31 Data - Panel 5.

125
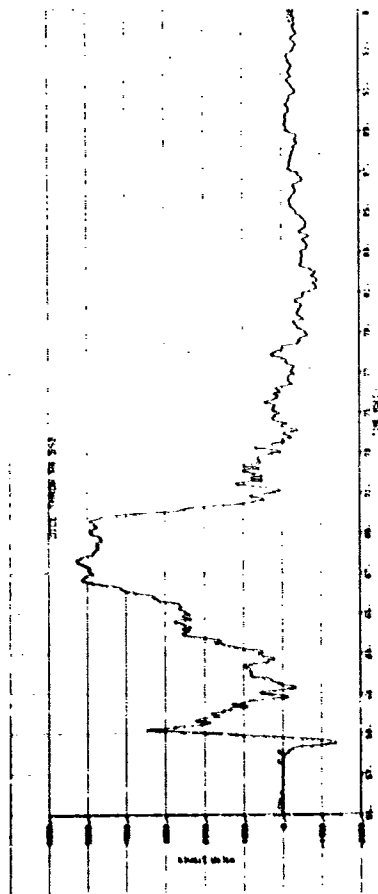
FIGURE 78. Strain Gage No. 43 Data - Panel 5.

126
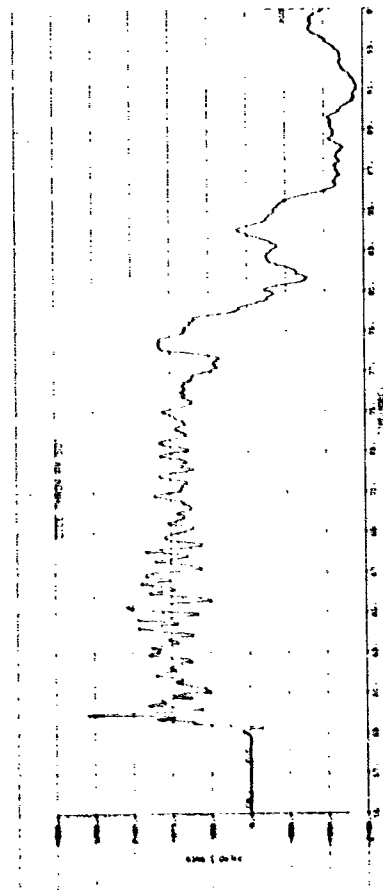
FIGURE 79. Strain Gage No. 1 Data - Panel 1.
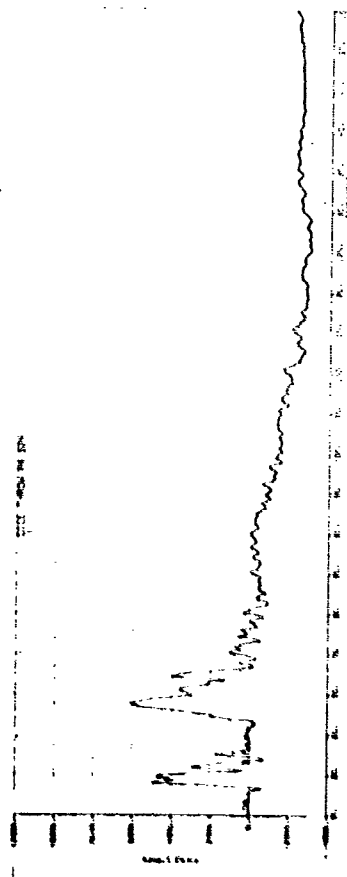
FIGURE 80. Strain Gage No. 24 Data - Panel 4.

128
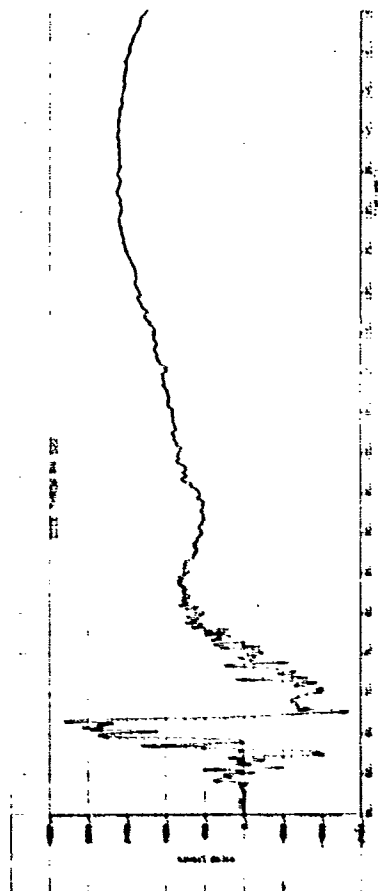
FIGURE 81. Strain Gage No. 22 Data - Panel ↗.

at approximately 9-10 ms after the blast intercept. Th
pattern occurred for the majority of gages installed on .hc
fuselage, and it is attributed to the synergistic effects
of overpressure and gust. It was found to manifest itself
as early as 7 ms after the blast intercept, which approxi-
mately corresponds to the fuselage natural frequency deter-
mined in the ground vibration tests conducted by the
manufacturer, Ref. 8.

The gust effect may, as in the case of gage 23, Fi₁
become increasingly prcnounced so as to reach the maximu
tensile strain value after the high frequency response h s
virtually died out. There, the gradual build up of tens
strains peaked between 70 to 8C ms after the shock arriʸ ₁

The correlation of the experimental data and the anaʼ ic
predictions obtained from the NOVA 2 solutions waf approu   d
with the understanding that the idealized modeling of pan
elements can, at best, represent a reasonable approximati
of the aircraft structure response. Certain limitations
the NOVA 2 modelirg are generally recognized and this test
offered a unique opportunity to put these limitations in
proper perspective.

The panel boundary condition representation in NOVA 2
forces a choice between either the simply supported or
clamped edge boundary conditions. This immediately suggests
a potential source of error. Similarly, the lack of pro-
visions for adequate modeling of a structural element in
conjunction with the adjoining structure constitutes a
departure from the true-to-life structure representation.

The NOVA 2 code was designed to predict the structural
response restricted to the overpressure effects. Therefore,
in the situations where the synergistic effects of over-
pressure and gust are present, the gust effects have to be
assessed separately and suitably combined with the overpres-
sure effect predictions.

The NOVA 2 assumptions of a single instant for the time
of shock arrival and of the overpressure uniform across
the entire panel surface (overpressure changing with time,
of course) are found to be inconsistent with the actual
events. Furthermore, the overpressure historics obtained
during the test differed significantly from the analytic
blast model incorporated within the NOVA 2 program. This

130
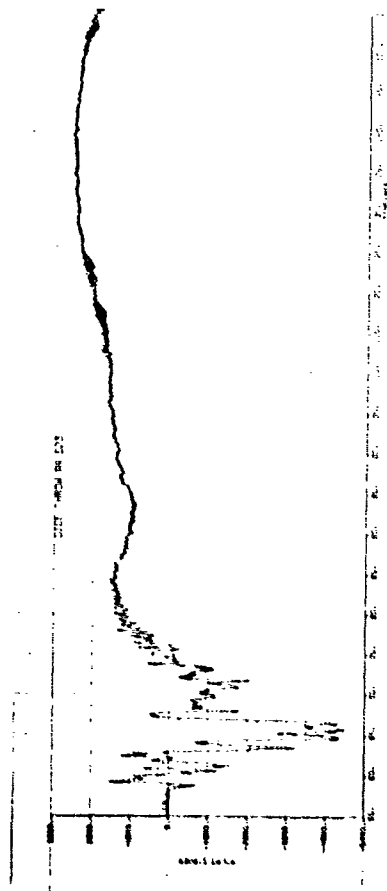
FIGURE 82. Strain Gage No. 23 Data - Panel 4.

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is illustrated in Figs. 68 through 72 which show super-imposed overpressure versus time plots for the analytic and experimental data. All these factors are bound to affect the accuracy of the analytic predictions.

Nevertheless, for all these limitations, NOVA 2 offers the panel response predictions which compare quite favorably with the experimental data. In some cases, these predictions may be remarkably accurate. In other cases, appropriate interpretation must be applied to the analytic data in order to define the sources of discrepancies and assess the significance of the analytic solution in relation to the true-to-life structural response.

These cases are graphically illustrated in the analytic and experimental strain data plots for panels 1, 4, 5, 6, and 8 presented in Figs. 40 through 67 and in Table 9 which summarizes the peak values.

For the center of Panel 1, the maximum strain values obtained from the NOVA 2 DEPROP, KADBOP, and KADBOB solutions varied between 3500 and 4100 microstrains and were comparable with the experimental strain value of 3400 microinches. However, the analytic strain histories, shown in Fig. 40, differ significantly from the experimental strain histories inasmuch as the dominant membrane strains indicated by the analysis contrast the test data which showed very significant bending strains.

Even greater discrepancies can be observed for the strains near the longer side boundary of the panel. The analytic strains of 10,100 and 11,100 microstrains were obtained from the NOVA 2 DEPROP and KADBOP solutions, respectively. The higher strains obtained from the KADBOP solution illustrate the effect of the more severe experimental overpressure as compared to the analytic blast used in the DEPROP solution, Fig. 68. These strain levels, which signify considerable yielding of the panel skin material, appear to be much greater than the corresponding experimental strains which peaked at 2880 microstrains, well within the elastic range. This discrepancy between the analytic and experimental values is further accutuated by the KADBOB solutions which use a beam representation of panels in the analysis. This representation was advertised as an acceptable approximation which offers considerable saving in the computer time without significant sacrifice of the solution accuracy. The colossal strain values of 45,600 and 70,900 microstrains obtained from the KADBOB-A

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TABLE 9. Correlation of the Analytic Strains Obtained from NOVA 2 Solutions and the Experimental Strains: Skin Panels - Maximum Response Data Summary.

Skin Panel No.	Location	Maximum Strain (Microstrains) at Time (milliseconds) After Shock Arrival				
		Experimental Data	DEPROF Solution	KADBOP Solution	KADBOB Solution	KADBOB-A Solution
1	Center	3400 @ 1.1 ms	3600 @ .5 ms	4100 @ .5 ms	4000 @ .5 ms	3500 @ .5 ms
1	Longside	2880 @ 1.8 ms	10,100 @ .6 ms	11,100 @ .5 ms	70,900 @ .6 ms	45,600 @ .6 ms
4	Center	5000 @ 1.0 ms	3700 @ .4 ms	3300 @ .4 ms	3300 @ .4 ms	-
4	Longside	6000 @ 1.3 ms	10,300 @ .4 ms	7500 @ .4 ms	9000 @ .4 ms	-
5	Center	-	5800 @ .5 ms	-	3600 @ .6 ms	-
5	Longside	8800 @ 1.5 ms	13,600 @ .5 ms	-	51,600 @ .7 ms	-
6	Center	3200 @ 0.6 ms	3100 @ .3 ms	-	3200 @ .4 ms	-
6	Longside	7600 @ 0.6 ms	10,500 @ .4 ms	-	27,200 @ .5 ms	-
8	Center	3800 @ .8 ms	-	-	2800 @ 0.7 ms (clamped edge model) 3200 @ 0.6 ms (SS edge model)	
8	Longside	2400 @ .9 ms	-	-	7900 @ 0.6 ms	-

NOTE: Unless stated otherwise, panels are modeled for clamped edge boundary conditions.

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and KADBOB solutions, repectively, (Figs. 56 and 57, and Table 9) clearly indicate that for strain levels beyond the elastic limit, the beam representation of panels produces increasingly unrealistic results. This was consistently demonstrated in the KADBOB solutions for panels 5 and 6, Figs. 62 and 64, respectively, with the corresponding strain maxima of 51,600 and 27,200 microstrains. As expected, for the larger of the two panels, panel 5, farther excursion into the plastic range was predicted.

The selection of panel 8 was based on the premise that this panel was most likely to demonstrate a typical response for the simply supported edges boundary condition. This access door panel was characterized by skin discontinuities at all edges. This was largely verified by the experimental data, Table 9, which indicated 3800 microstrains maximum at the center of the panel and lower strain, 2400 microstrains, near the panel edge.

Two analytic solutions were obtained for this panel. For one solution, the panel was modeled as simply supported, and for the other solution as clamped-edge boundary conditions. The resulting analytic strain data and the corresponding experimental strain data plots are presented in Figs. 65 through 67. The experimental strains obtained for the inside surface at the center of the panel corresponded rather closely to the analytic strains for the simply supported edge model, Fig. 65. A disparity can be observed between the outside surface strains for the experimental and analytic data. This mode of behavior, mentioned earlier in the discussion of panel 1, implied that, in the analysis, panels are treated as membranes rather than plates. The experimental data indicated the panel response more closely resembles that of a plate. It should be noted that the membrane behavior is related to the panel width-to-thickness ratio and to the load level. Thus, thin panels tend to respond as membrances and, at higher load levels approaching the yield point and beyond, the membrane action becomes increasingly dominant.

The clamped-edge solution for panel 8, Figs. 66 and 67, shcwed considerably greater discrepancy between the analytic ana experimental data, particularly near the panel edge. The analytic strain value at that location, 7900 microstrains, implied a response ;in the plastic range whereas the experimental strain maximum of 2400 microstrains clearly denoted a response within the elastic range.

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Overall, Method 3 does an acceptable job of approxi-
mating complex real-world panel structures. However, con-
siderable judgement is required on the part of the analyst.
The engineering time required to obtain a first-case solution
is approximately 40 man-hours and the computer time may
exceed 2 hours on a large-scale computer (CDC-6600 was used
in this analysis).

Longerons

Because of recording channel limitations allowed for
longerons 4 and 5, the instrumentation was clearly inade-
quate for a comprehensive definition of the longeron
response during the overpressure phase of the blast. This
inadequacy was further compounded by erratic behavior of
the deflection gages and the crucial strain gages Nos. 56
and 59, (Fig. 12) which were located on the outside surface
of the skin directly opposite the longeron flanges. Strain
gages 55, 57, and 58 (Fig. 12), located on the longeron
legs, were too close to the neutral axis to provide data
relevant for correlation with the analytic data. The
longeron leg which terminated in a bulb-type flange was
unsuitable for strain gage installation.

The only strain gage which appeared to provide
reasonable data was gage 59. Unfortunately, the tensile
strain indicated by this gage could not be readily accepted
as valid for the location where compressive strain was
expected, based on the load configuration and structure
geometry.

A plausible, though possibly conjectural, explanation
for the tensile strains actually developed at the gage 59
location is postulated as the overall response of the
fuselage curved panel sector bounded fore and aft by two rigid
bulkheads at Stations 262 and 306, respectively, and, top to
bottom, by longeron 6 and the lower longeron. In the vicinity
of STA 306, at the outside surface location, this large fuse-
lage sector might well be subjected to the initially tensile
stresses as indicated by strain gage 59. This line of reason-
ing leads to an observation that a structural member which
constitutes one element of a multi-element integral structure
should be modeled in conjunction with the entire structure
to enhance adequate correlation between the experimental
and analytic data.

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The longeron 5 analysis was accomplished using DEPROB
and KADBOB solutions. The former was based on the analytic
blast model and, for the latter, the experimental over-
pressure data of the closest pressure gage was used as the
input. In both solutions, the longeron was assumed clamped
at either end.

From the DEPROB solution (Fig. 83) the overall maximum
compressive strain of 4800 microstrains was obtained at the
longeron inner flange at the support point and the corres-
ponding outside flange strain was 2200 microstrains in
tension. At the longeron mid-span (Fig. 84), lower strains
of 2700 and 1200 microstrains were indicated for the inner
and outer flanges, respectively.

Slightly lower strains were consistently obtained from
the KADBOB solution (Figs. 85 and 86), because the experimen-
tal overpressure profile less severe than the analytic blast
model, Fig. 70.

The strain gage 59 data plot was included in Figs. 84
and 86 for reference only. The lack of adequate experimental
data precluded a correlation with the analytic predictions.
However, the available experimental and analytic data support
the observation that only moderate strains were developed in
the longerons during the overpressure phase of the blast.

Fuselage Frame

The fuselage frame STA 277, Fig. 87, is a complex,
partial ring structure which consists of several composite
elements with discontinuities along the perimeter. Presence
of elastic supports such as longerons and cantilever seg-
ments increases the complexity and makes the structural
modeling for the NOVA 2 solution difficult and requires a
series of approximations. In addition, the overpressure
representation which, in the case of a frame analysis, was
required to account for pressure variation around the frame
at any given time during the diffraction phase, involved
approximations inherent in the NOVA 2 blast model. Each
approximation carried with it a potential source of error
which was bound to affect the solution accuracy.

The frame analysis was restricted to a DEPROB solution.
The KADBOB solution which, at present, lacks the capability
for a multi-point representation of pressure variation
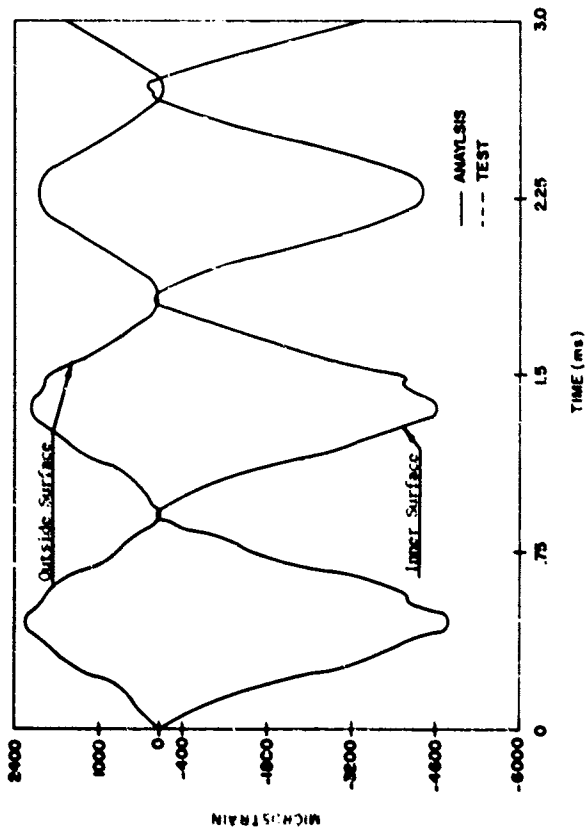
FIGURE 83.   Strain Data Longeron 5, Support
NOVA 2 - DEPROB Solution.
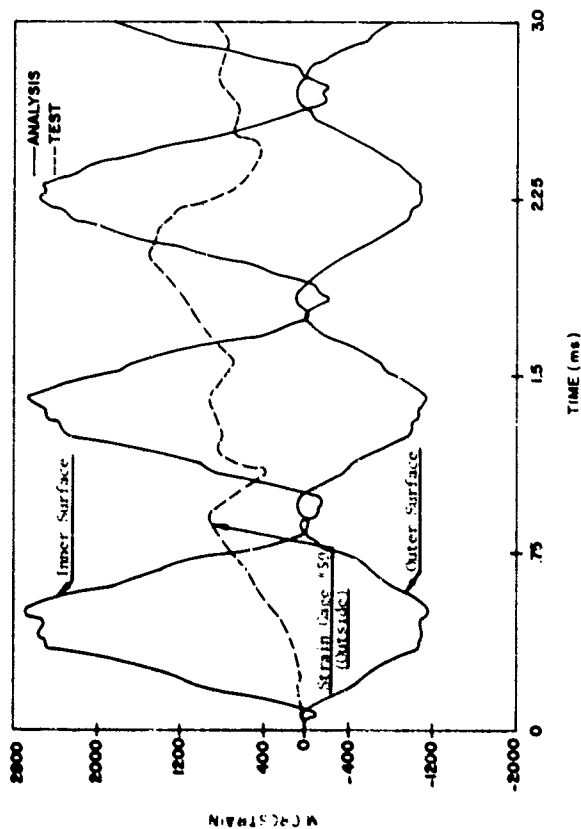
FIGURE 84. Strain Data Correlation - Longeron 5, Mid-Span, NOVA 2 - DEPROB Solution.
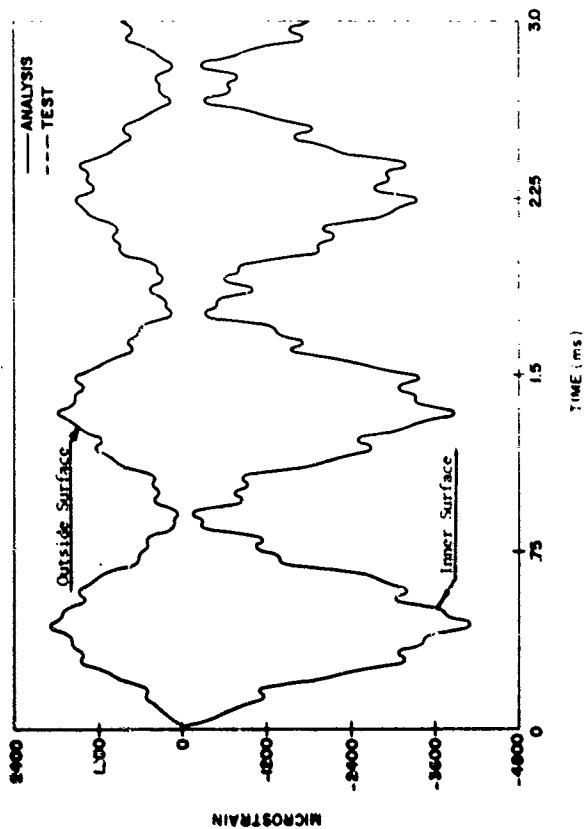
FIGURE 85. Strain Data Longeron 5, Support
NOVA 2 - KADBOB Solution.

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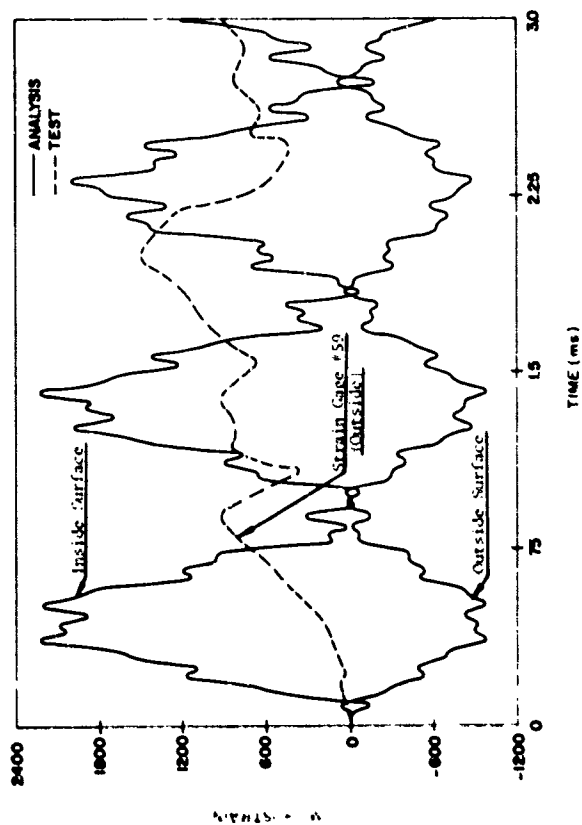
FIGURE 66. Strain Data Correlation - Longeron 5, Mid-Span, NOVA 2 - KADBOR Solution.

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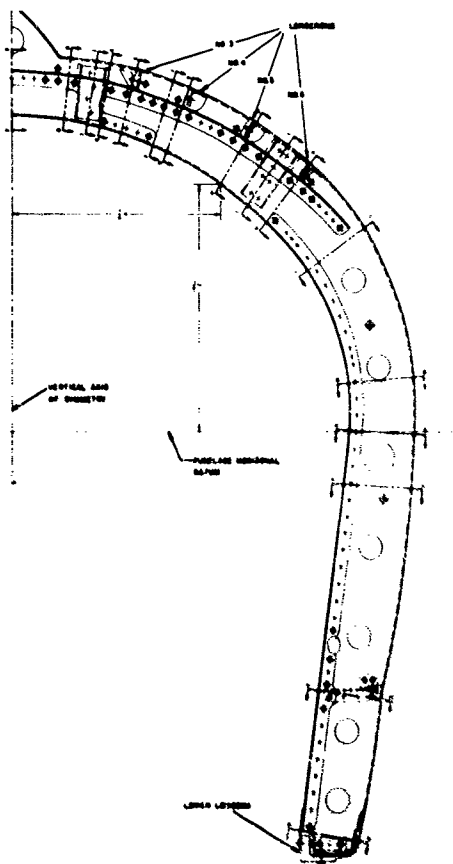
NOTE: Dimensions Listed in TABLE 10.

FIGURE 87. Fuselage Frame STA 277, Configuration.

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TABLE 10. Section Dimensions Data Fuselage Frame STA 277.

VIEW 1

VIEW 2
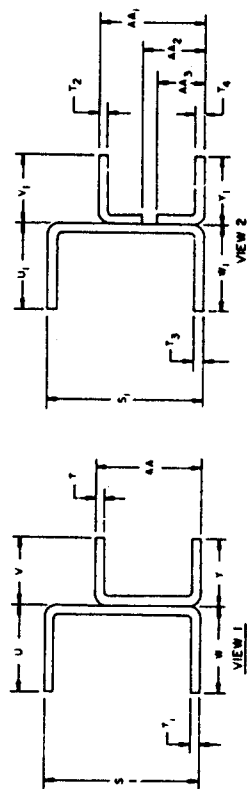
around the frame, was not attempted. (The code designers at Kaman AviDyne are currently working at improving NOVA 2 code to include that capability.)

The predicted strain data, together with relevant experimental data, are presented in Figs. 88 through 91. For the frame support at the lower longeron location, i.e., section RR in Fig. 87, high strains in excess of 14,000 microstrains were predicted, Fig. 88. It is difficult to assess the validity of this prediction because a realistic representation of this frame-to-lower longeron joint poses an intricate modeling problem. In view of the failure at this joint during the test, Fig. 25, close correlation between the analytic prediction and the test results is quite remarkable and may be a coincidence.

For the plane of geometric symmetry at section AA, Fig. 87, moderate strains, below 2800 microstrains, were predicted. A low peak strain (650 microstrains) was obtained experimentally by gage 48, Fig. 89. Modeling of the section AA was also complicated, particularly by a discontinuity in the main element of the frame and by the presence of an intersection with the vertical stabilizer structure. Representation of that configuration required a great deal of judgement which apparently produced fair correspondence between the predicted and experimental data.

For the curved segment of the frame, bounded by sections HH and MM, Fig. 87, the maximum predicted strains approached 4500 microstrains at the frame inner flange. The corresponding experimental strain maxima were 5500 microstrains for section HH, Fig. 90, and 2000 microstrains for section MM, Fig. 91. Since all these strains were compressive, a possibility of buckling failure could not be ruled out. In fact, an inner flange buckling in that region occurred in the free-standing aircraft BUNO 145062, Fig. 30. Detailed examination of the failure area revealed a lightening hole in the frame web of BUNO 145062 at that location. In the instrumented aircraft, BUNO 149558, the lightening hole was absent. It appears that this segment of the frame was critical and a relatively small strength reduction in this area precipitated a failure.

Considering the approximation procedures involved in modeling the frame structure, the remarkably close correlation between the predicted data and test results is regarded as highly satisfactory.

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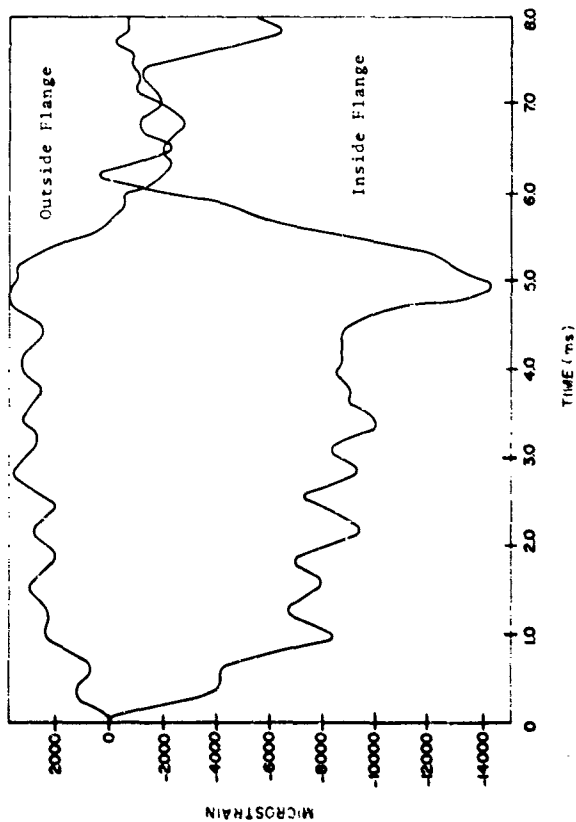
FIGURE 88. Predicted Strains at Frame STA 277 Support,
Section RR (Fig. 87, Ref.).
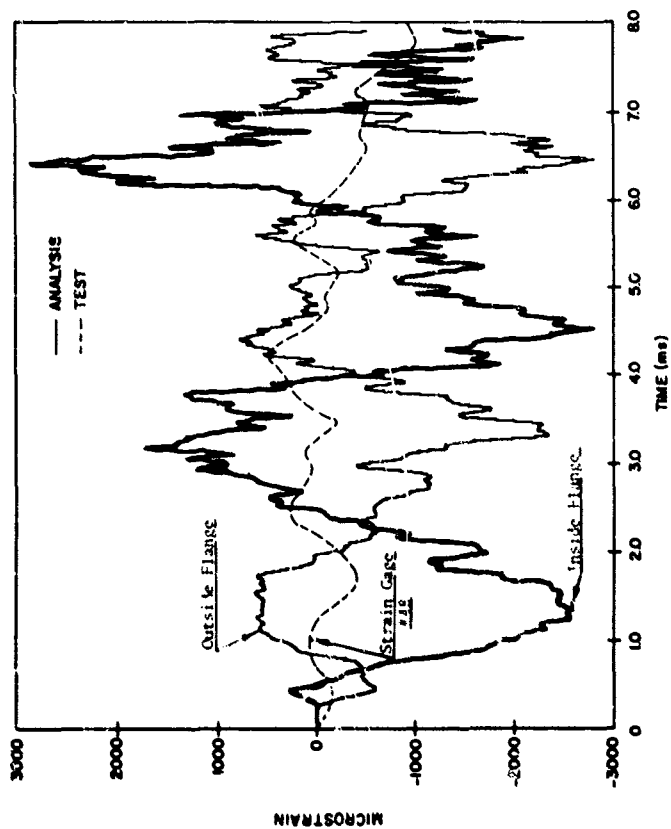
FIGURE 89. Strain Data Correlation Fuselage Frame
STA 277, Section AA.

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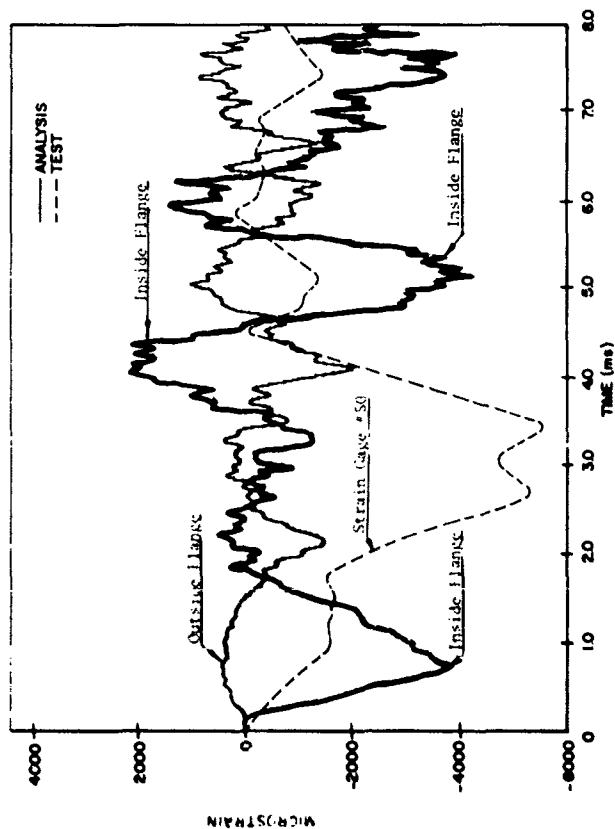
FIGURE 90. Strain Data Correlation Fuselage Frame
STA - - , Section HH.
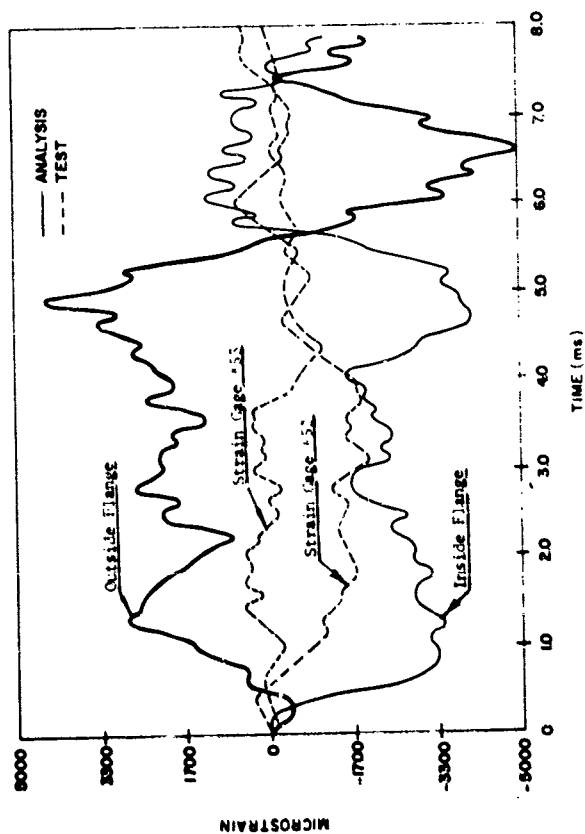
FIGURE 91. Strain Data Correlation Fuselage Frame
STA 277, Section MM.

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AIRCRAFT RIGID BODY MOTIONS

All three aircraft had motion picture cameras located
as shown in Fig. 1. The film record obtained for the free-
standing aircraft BUNO 145062 was of poor quality and inade-
quate for analysis.

Mapping of rigid body motions during the positive and
negative phases of the blast was, therefore, limited to BUNO
149558 and 145074. Since only one camera was installed for
each aircraft, the rigid body motions which could be deter-
mined with a reasonable degree of confidence were restricted
to:

1. Lateral translation.

2. Roll.

3. Yaw.

The analysis of the film records, Ref. 9, extended to
the point in time when the field of vision became obscured
by the cloud of dust raised by the blast. For aircraft BUNO
149558 and 145074, this corresponded to, respectively, 0.8
and 0.55 s after the shock front arrival.

Results of this analysis are presented in Figs. 92
through 97. Because of the tiedown restraints (primarily
against the translation away from GZ), Fig. 4, the aircraft
motions during the positive blast phase were relatively
small (translation) to virtually nonexistent (roll). Size-
able motions occurred during the negative blast phase. BUNO
149558 experienced, approximately, 0.9 m (2.9 ft) maximum
translatory motion of the aircraft CG, Fig. 92 towards the
GZ. The corresponding rotational motions in roll and yaw
peaked at 6 and 18 degrees, respectively, Figs. 93 and 94.

Generally smaller motions were obtained for BUNO 145074
with 0.5 m (1.6 ft), 4.5 degrees, and 16 degrees indicated
for the translation, roll and yaw, respectively, Figs. 95
through 97. The smaller motions for this aircraft which was
exposed to 62.1 kPa (9 psi) overpressure as compared to BUNO
149558 which experienced 41.4 kPa (6 psi) overpressure can
be explained by the fact that, following the catastrophic
fuselage failure, Fig. 18, the aircraft area exposed to the
effect of the negative drag phase was greatly reduced.

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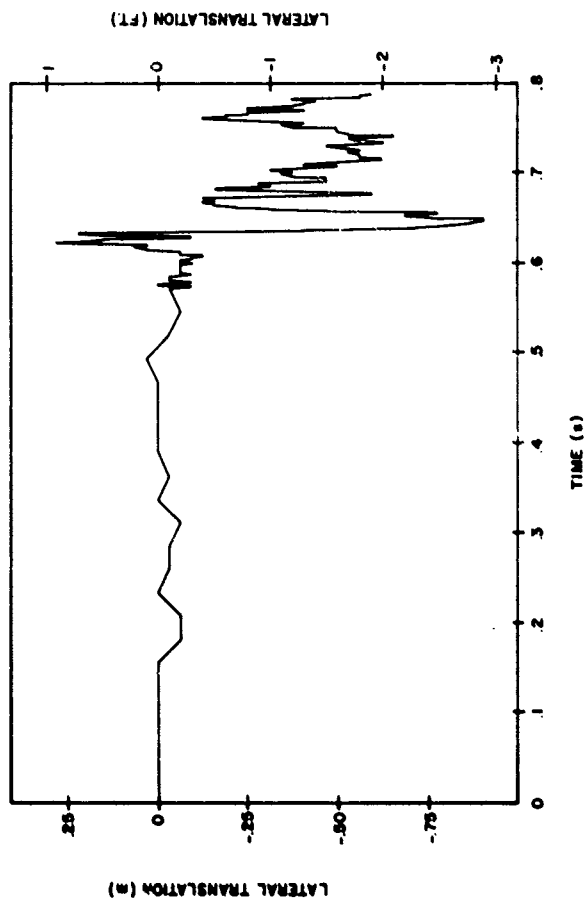
FIGURE 92. Lateral Translation at Aircraft CG - BUNO 149556.

149
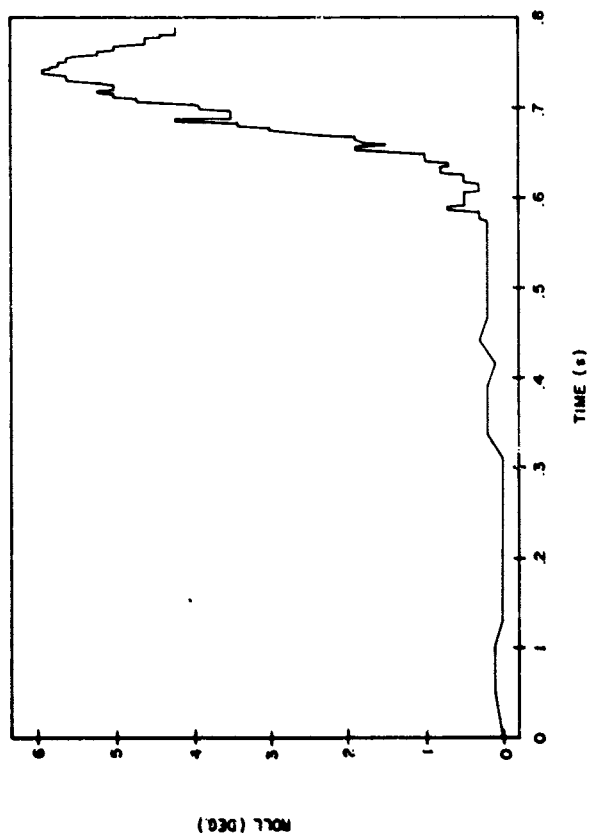
FIGURE 95. Roll of A/C BUNO 149558.

150
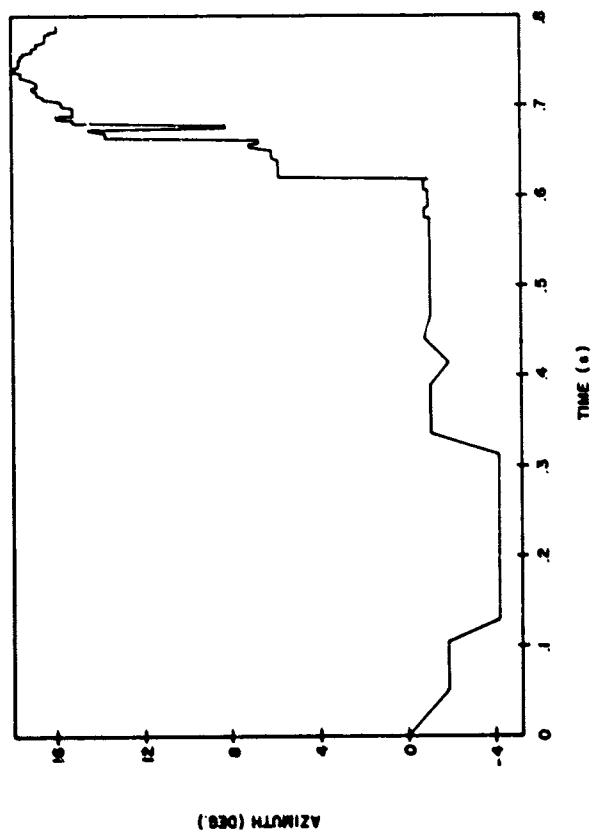
FIGURE 94. Yaw of A/C BUNO 149558.

AZIMUTH (DEG.)

TIME (s)

151
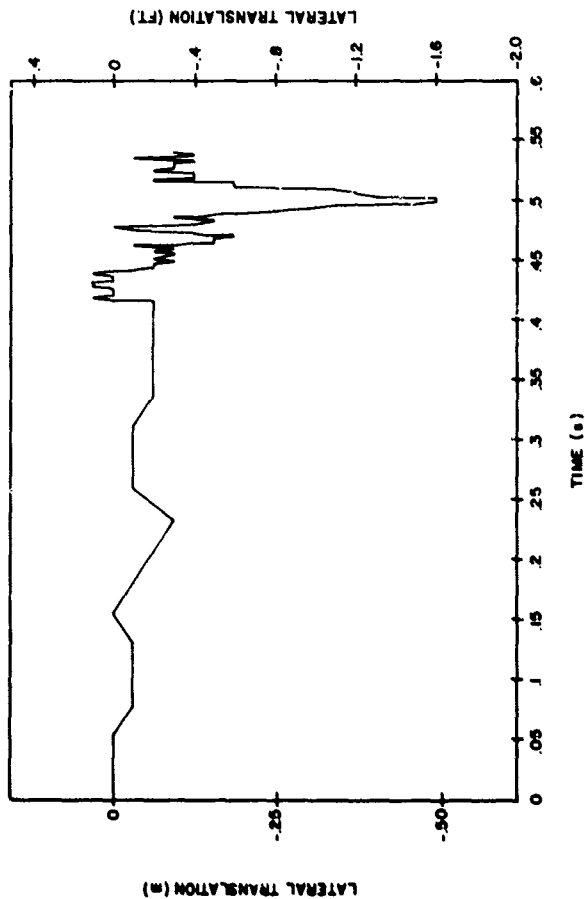
FIGURE 95. Lateral Translation at Aircraft CG -
BUNO 145074

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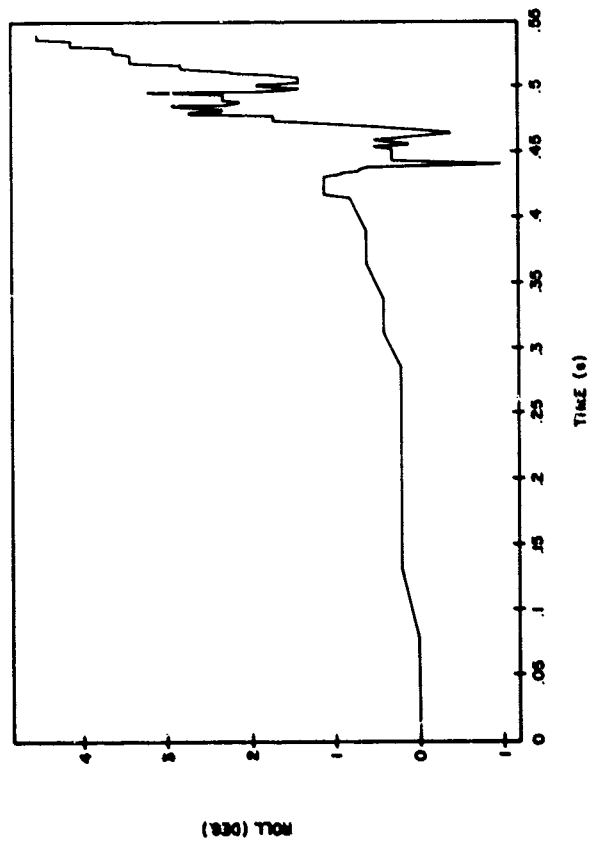
FIGURE 96. Roll of A/C BUNO 145074.

153
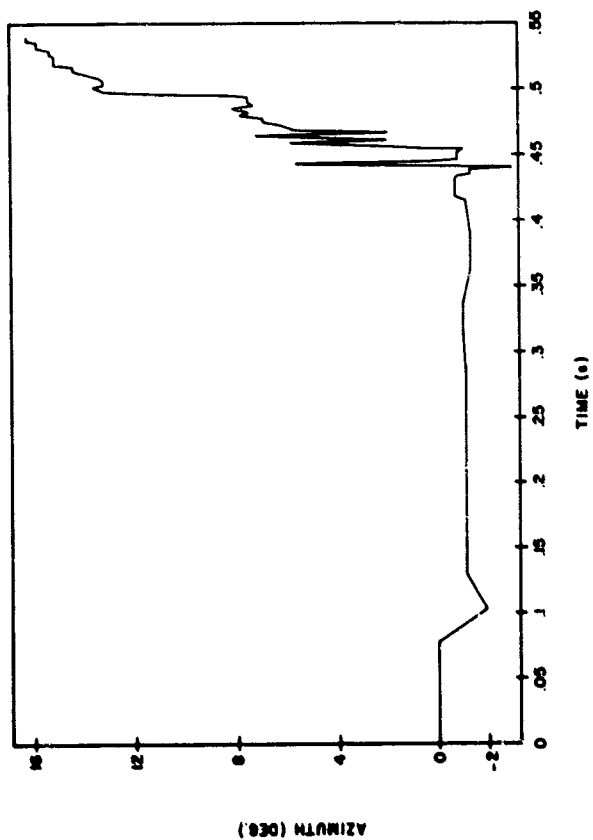
FIGURE 97. Yaw of A/C BUNO 145074.

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in absence of the quantitative data for the free-standing aircraft, BUNO 145062, qualitative observations will have to suffice. This aircraft did not exhibit any tendency to overturn during the positive drag phase but the tire skid marks provided valid evidence that translatory and yaw motions did take place. The sequence of events was difficult to reconstruct from the postblast location of the aircraft which showed:

1. The main landing gear wheels were found displaced approximately 30 cm (12 in.) downstream (away from the GZ). Tire marks extending 30 cm (12 in.) forward were also observed.

2. The nose landing gear wheel was found displaced 38 cm (15 in.) upstream (towards the GZ) with no indication of forward or aft motion.

As mentioned earlier, the tiedown system used for the two secured aircraft provided an effective restraint against aircraft motions during the positive blast phase. The extent of that effectiveness is illustrated by the data on energy absorbed immediately following the shock intercept by the load cells incorporated in the tiedown system, Table 11.

As an initial attempt at defining aircraft rigid body motion during blast phase, this test provided basic data for follow-on studies in the verification of overturning predictions.

NUCLEAR TEST DATA CORRELATION

A review of old nuclear test data reveals that project 3.1 in OPERATION SNAPPER in the spring of 1952 involved extensive testing of parked aircraft. Test films and photographic records were reviewed to check correlation with the recent DICE THROW results. Numerous aircraft were exposed to combined blast and thermal effects at different intensities and from different orientations. The aircraft of interest were the F-47, the XF-90 and the F-86.

The damage to these aircraft exposed to increasing pressure levels was observed as follows:

Onset of damage pressure - slight buckling of control surfaces, access doors blown off, and occasional panel ruptured.

TABLE 11. Energy Absorption by the Tiedown Load Cells
            Following Blast Intercept.

Air-craft No.	Load Cell No.	FORCE		STROKE		ENERGY ABSORBED	
		kN	lbf	m	ft	kJ	ft-lbf
BUNO 149558	1	46.7	10,500	.0190	.063	0.896	661
	2	46.7	10,500	.0016	.001	0.0149	11
	3	46.3	10,400	.0538	.177	2.496	1841
	4	46.7	10,500	.0635	.208	2.967	2188
	5	47.8	10,750	.0286	.094	1.367	1008
	6	49.8	11,200	0	0	0	0
BUNO 145074	1	47.8	10,750	.0079	.026	0.377	278
	2	47.8	10,750	.0064	.021	0.317	234
	3	46.3	10,400	.1286	.422	5.951	4389
	4	45.1	10,150	.1492	.430	6.737	4969
	5	46.7	10,500	.0778	.255	3.635	2681
	6	45.1	10,150	.0540	.177	2.436	1797

NOTE: Details of the tiedown system geometry are available
        at NWEF AT Department.

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Intermediate pressure - moderate buckling of control surfaces and fuselage panels between substructure supports, broken canopies (on F-47 only). Damage appears much like that occurring on the two A-4C aircraft subjected to 6 psi in the DICE THROW test.

Severe damage pressure - large rigid body motions causing major substructure damage, vertical stabilizers broken, canopies broken, XF-90 and F-47 fuselages broken completely at aft section joint - - damage similar to A-4C damage at 9 psi location in DICE THROW.

Admittedly, thermal radiation effects played a big part in the total picture of damage; i.e., many aircraft had scorched paint, wrinkled skins and extensive damage to the very vulnerable control surfaces. However, the damage described above was, for the most part, attributed to blast effects. A conscious effort was made, in the present assessment, to consider only blast induced damage; nevertheless, some synergistic effects are necessarily present.

Overall, this data correlated well with the results on the A-4Cs in DICE THROW. The presence of skin panel dishing-in or buckling was similar and tail cone buckling was quite similar. Both sets of test data confirm that the weak point in the fuselage is at the aft section or tail joint. However, even though the SNAPPER data shows bent rudders, they were not torn off the tails as was uniformly the case in DICE THROW. The rudder hinges must be extremely weak on the A-4C aircraft.

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CONCLUSIONS

The conclusions, presented below, are based on the experimental and analytic data for the Navy A-4C aircraft. It should be noted that the structural response to over-pressure is directly related to the aircraft design characteristics. An extension of these conclusions to aircraft of difference class, configuration, etc. can only be made with proper regard to the design and configuration similarities.

The conclusions are listed grouped into four separate categories:

    1.   General - Test Data

    2.   Method 1 - Assessment

    3.   Method 2 - Assessment

    4.   Method 3 - Assessment

GENERAL - TEST DATA

1.   The response of panels in the actual aircraft structure does not conform to the classic, clamped edge boundary condition. An intermediate response between the simply supported and clamped edge boundary conditions is implied by the test data. The analytic representation (NOVA 2) allows only one or the other.

2.   Location of the panel boundary could not be determined precisely because it depended on substructure configuration and rigidity. With a finite rigidity of the true-to-life substructure, the effective panel boundary location was a function of load intensity. None of the analytic methods allows for boundary flexibility.

3.   Because of the physical size of strain gages, their strain averaging characteristic makes the determination of maximum strain subject to significant errors in the regions of steep strain gradients.

4.   Rigidity of panel boundaries is shown to be a dominant factor in panel response characteristic. This was evidenced by the response of panels 1 and 8 whose behavior approached that of panels with a simply supported edge boundary condition. This is contrasted with the behavior of panels 4, 5, and 6 which behaved more like panels with a clamped edge boundary condition. The former panels were located on vertical stabilizer characterized by more flexible substructure. The latter were located on the fuselage with more rigid substructure.

5.   The likely effect of finite rigidity of the substructure at panel boundaries is that, at lower stress levels, the panel response resembles more closely the response for a clamped edge boundary condition. At higher stress levels, the panel response tends to approach the response for simply supported edge boundary condition.

6.   Only narrow limits separate slight damage from catastrophic failure of the aircraft structure exposed to the blast test conditions. In terms of the range for the parked and tied down aircraft, this amounts to approximately 60 m (200 ft).

7.   Local substructure failure may lead to a premature skin panel failure.

8.   Holes in the substructure members act as fracture initiation points and are likely causes of premature failure.

9.   Substantial pressure variations across structural elements such as skin panels are experienced during the blast diffraction phase.

10. Time of shock arrival for panel elements varies from point to point and is dependent on the panel shape and its orientation with respect to blast.

11. The strain history for flat panels varies across the panel area. Strains at the center of the panel peak to a maximum value during the first oscillation which is followed by attenuation of strain amplitudes during subsequent cycles. At the panel edge, several cycles with increasing amplitudes precede the attenuation trend.

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12. The strain history for curved panels is more uniform
across the panel area. Maximum response during the first
oscillation, followed by an attenuation, is generally observed
at the center as well as at the edge of the panel.

13. Strain histories for fuselage locations were characterized
by initial high frequency oscillations, followed by low
frequency oscillations, which resulted in an even higher,
delayed, secondary peak.

14. For gages on panels 4 and 5 (curved panels located at
approximately mid-fuselage), the maximum strain values are
indicated at approximately 10 ms after the shock intercept.
This implies a synergistic effect of overpressure and gust
and constitutes a convincing evidence that these two effects
can not be divorced from one another in considering the
overall vulnerability of aircraft structure to blast.

15. The maximum response of panels to overpressure alone
was generally observed to occur within 1 ms after the blast
intercept.

16. The synergistic effects of overpressure and gust first
appear approximately 7 ms after the shock intercept. This
corresponds approximately to the fuselage natural frequency,
demonstrated in ground vibration tests conducted by the
manufacturer.

17. Gust effects were observed to become increasingly more
prominent and reached their maximum at 70 to 80 ms after the
shock intercept. For strain gages installed on the fuselage
in the direction parallel to aircraft axis, the overall
maximum response was obtained at that time.

18. The strain gage data survey for the strain gradients
developed in the vicinity of the panel boundary along the
longer edge further reinforce the conclusion that the actual
panel response falls between the clamped and simply supported
edge boundary conditions. For the simply supported edge
boundary condition, a low strain gradient is expected with
membrane strains dominating. For the clamped edge boundary
condition, a very high strain gradient is predicted with
bending strains dominating. The experimental data also show
high strain gradients and significant bending strains in the
vicinity of the panel boundary. However, the differences
between strains at panel center and at the edge are not as
great as expected for the theoretical clamped edge boundary
condition.

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METHOD 1 - ASSESSMENT

1.   Critical overpressure calculated at 17.2 kPa (2.5 psi) for the sure-safe condition according to Method 1 is slightly conservative for fighter and fighter-bomber class aircraft represented by the A-4C.

2.   Critical overpressure calculated at 137.9 kPa (20 psi) for the sure-kill condition according to Method 1 grossly exceeds the limit of the attack-class aircraft survivability. An A-4C sustained catastrophic failure at a 62.1 kPa (9 psi) peak free-field overpressure.

3.   Vulnerability envelopes constructed according to Method 1 appear rather crude. The multiple ground range values obtained for a given height of burst are unrealistic and tend to be confusing.

4.   Method 1 provides means for rapid determination of the vulnerability envelopes and can be satisfactorily used where the range error factor of two is acceptable.

5.   The engineering time required to obtain a solution according to Method 1 is assessed at 20 minutes.

METHOD 2 - ASSESSMENT

1.   The panel analyses according to Method 2 yield results which may differ significantly from the experimental data. However, the discrepancies are found to be within the error limits specified for this method.

2.   The results of frame analysis according to Method 2 were found to be compatible with the test results.

3.   The longeron analysis according to Method 2 for sure-safe and sure-kill conditions may lead to contradictory results, i.e., indicated critical overpressure value calculated for the sure-safe condition may be higher than the overpressure value for the sure-kill condition.

4.   Considerable familiarity with aircraft structural response to blast and experience in structural modeling are required to ensure rational results from the analysis using Method 2.

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5.   The aircraft vulnerability data obtained according to
Method 2 are well within the 1.6 error factor limits speci-
fied in terms of the equivalent critical range values.

6.   Method 2 is distinctly superior to Method 1. Selection
of weakest structural elements for analysis and their model-
ing in some detail assures results of greater accuracy.

7.   Analytic solutions based on Method 2 can be obtained in
less than ten seconds of computer time using OVPR2 code.
The engineering time required to obtain a solution is assessed
at five hours for the first case and approximately two hours
for each subsequent case for the same aircraft.

METHOD 3 - ASSESSMENT

1.   The NOVA 2 computer code represents by far the most
sophisticated analytic tool of the three methods considered.

2.   NOVA 2 offers a capability for modeling structural
elements in considerable detail. This ensures a high degree
of accuracy for predicting the overpressure response of an
idealized, isolated structural element.

3.   There are no provisions in NOVA 2 for adequate modeling
of structural elements in conjunction with the adjacent
structure. This tends to reduce the accuracy of NOVA 2
predictions.

4.   NOVA 2 is limited to the analysis of overpressure
effects on aircraft structure. Consideration of synergistic
effects of overpressure and gust are beyond its scope.

5.   The assumption in NOVA 2 that a given time of the shock
arrival applies to the entire area of the element analyzed
holds true only in specific cases. A finite time is required
for the shock to sweep across the structure in the majority
of situations. This introduces some error to the analytic
solution.

6.   Complex shock reflection patterns are evident from the
experimental data but are not duplicated in the analytic
(NOVA 2) blast representation.

7.   For the structural elements of considerable size,
particularly the curved elements such as fuselage frames,
NOVA 2 contains no provisions to adequately model the

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overpressure distribution during the diffraction phase. The assumption of constant pressure across the element analyzed by NOVA 2 does not necessarily represent the actual condition and constitutes a source of error in the analytic prediction. In actuality, pressure gradients do exist across the structure.

8. By providing data for idealized structural elements analyzed under selected boundary condition, NOVA 2 is capable of approximating the response of a true-to-life structural element.

9. Weighing all the evidence, boundary condition modeling is most critical for panel response prediction.

10. Panel 8 modeling with the simply supported edge boundary condition provided results closely comparable with the experimental data--at the panel center 3180 versus 3800 microstrains and at the edge 2300 versus 2400 microstrains were indicated for the analytic (NOVA 2) and experimental values, respectively.

11. Panel 8 modeling with the clamped edge boundary condition representation (NOVA 2) predicted considerable yielding of the material at the panel boundary. This was not verified by the experimental data where the corresponding strains were well within the elastic limit.

12. Lack of adequate correlation between the analytic (NOVA 2) and experimental data is evidenced by predominantly membrane strains indicated at panel center in the analysis while the experimental data show bending strains as dominant.

13. In some cases, strain values obtained from the NOVA 2 (KADBOB) solution are extremely sensitive to the relationship between the numerical integration time step, DELTIN and the segment length parameters. For short segment lengths, small DELTIN values are required. If DELTIN value is insufficiently small, unrealistically high strain values are obtained. This was apparent in panel 8 analysis for the simply supported edge condition, but was not indicated in the solutions for panels 1 and 5 with the clamped edge boundary condition representation.

14. Analytic predictions based on beam representation of panels (NOVA 2 - KADBOB) give increasingly poor predictions of peak strain for increasing excursions in the plastic range. For conditions beyond the material yield, KADBOB solutions indicate colossal strains at the panel boundary, completely unrelated to the experimental data. For panel 1, the analytic strains were 45 times higher than the corresponding experimental strains. This trend is not indicated by the panel, NOVA 2 - DEPROP, representation.

15. Applicability of beam representation for panels is not clearly specified in the Method 3 (NOVA 2) handbook, and its accuracy is not defined as related to the stress range. It appears that the stress level is the factor governing the solution accuracy.

16. Taking a holistic view, NOVA 2 does an acceptable job of approximating true-to-life structural response. However, this requires considerable engineering time and expert judgement on the part of the analyst. The engineering time required to obtain a solution is assessed at 40 man-hours for the first case, and the required computer time may exceed two hours on the CDC 6600 computer system.

17. This experimental study, exclusively devoted to the primary airframe structural response to overpressure loads, was not concerned with the nonstructural damage. Loss of rudders, damage to landing gear doors, and rupture of access doors in the engine area were all sustained during the test. This damage was in itself serious because of potential flight impairment and/or fire hazard and should be considered in the overall assessment of aircraft vulnerability/survivability.

18. A brief review of the nuclear test data obtained during OPERATION SNAPPER in 1952 indicated that, for the attack class aircraft, the damage at corresponding overpressure levels was comparable to that sustained by A-4C aircraft during the nonnuclear DICE THROW event.

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APPENDIX A

LOADING AND RESPONSE PREDICTIONS FCR TESTING AN F-4
TRAILING EDGE FLAP IN PRE-DICE THROW II EVENT

Abstracted from report prepared by John M. Calligeros
and William N. Lee of Kaman AviDyne, Burlington, Nassachusetts
01803 under DNA Contact No. DNA001-75-C0080
August 1975

OBJECTIVES

The purpose of this technical memorandum is to provide
pretest support to the Naval Weapons Evaluation Facility
(NWEF) for testing a F-4 trailing edge flap in the PRE-DICE
THROW II project. This support includes the following
items:

1. Calculation of the flap panel response and the
likelihood of rib buckling to blast using the latest revision
of the NOVA code.

2. Review of the strain gage and pressure gage instru-
mentation scheme formulated at NWEF and making recommendations
for setting instrumentation gains.

3. Definition of the test data-reduction requirements.

DESCRIPTION OF STRUCTURE

The flap assembly is illustrated in Figs. A-1 through
A-3. Figure A-3 also shows strain gage locations. The
structural items of interest in this analysis are a typical
skin panel 1.02 mm (0.040 in.) thick, measuring 495 x 133 mm
(19.5 x 5.25 in.) between rivet lines, as shown in Fig. A-3,
and a rib (Fig. A-1). A detail of the rib, taken from
McDonnell-Douglas drawing 32-18509, is shown in Fig. A-4.
As indicated in Fig. A-2, the extremities of the panel are
represented by fastener lands 1.60 mm (0.063 in.) thick and
19.1 mm (0.75 in.) wide. The pertinent dimensional and
material characteristics of these two structural items are
summarized as follows:

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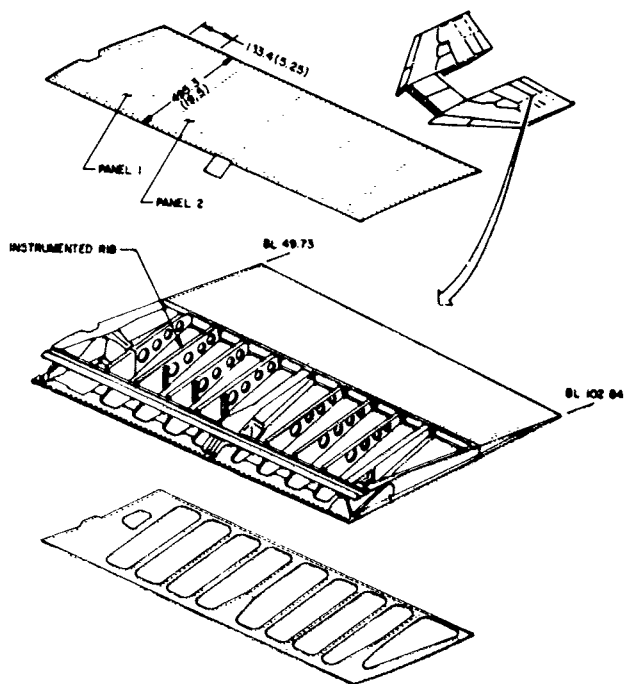
The length is in millimeters
with inches in parenthesis.

FIGURE A-1. Trailing Edge Flap.
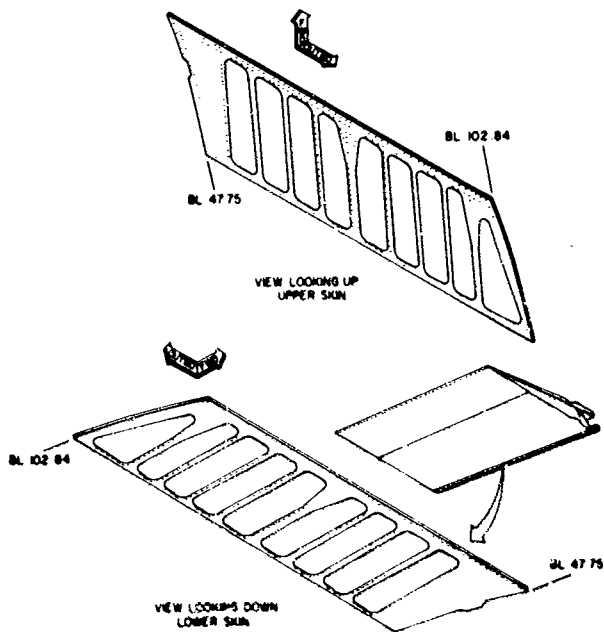
FIGURE A-2. Trailing Edge Flap Upper and Lower Torque Box Skin Diagram.
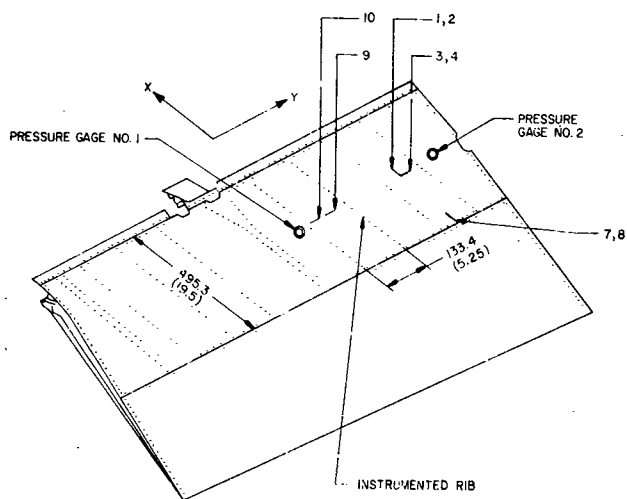
The length is in millimeters
with inches in parenthesis.

FIGURE A-3. Instrumentation Location Diagram.

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265.43
(10.45)

LOOKING INBOARD

FWD
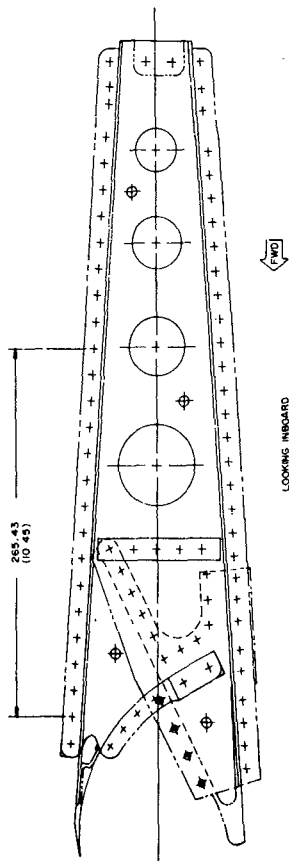
The length is in millimeters
with inches in parenthesis.

FIGURE A-4. Rib Assembly x_B 72.750.

Skin Panel

length = 495 mm (19.5 in.) between rivets

width = 133 mm (5.25 in.) between rivets

thickness = 1.02 mm (0.040 in.)

fastener lands thickness = 1.60 mm (0.063 in.)

fastener lands width = 19.1 mm (0.75 in.)

material 7075-T6 Aluminum

E (Young's modulus of elasticity) = 71.0 GPa
  $(10.3 \times 10^6$ psi)

f_{ty} (tensile yield stress) = 476 MPa
  (69,000 psi)

ν (Poisson's ratio) = 0.33

G (shear modulus) = 26.9 GPa $(3.9 \times 10^6$ psi)

ρ (density) = 2.77 Mg/m³ (0.10 lb/in.³)

Rib

thickness = 0.81 mm (0.032 in.)

r_1 = radius of 1st lightening hole
  $\approx$ 29.2 mm (1.15 in.)

r_2 = radius of 2nd lightening hole
  $\approx$ 21.3 mm (0.84 in.)

ℓ = distance between lightening hole centers
  = 88.6 mm (3.49 in.)

d = rib depth between lightening holes
  = 93.7 mm (3.69 in.)

material 7178-T6 Aluminum

E = 71.0 GPa $(10.3 \times 10^6$ psi)

f_{ty} = 503 MPa (73,000 psi)

ρ = 2.77 Mg/m³ (0.10 lb/in.³)

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The dimensions r_1, r_2, t, and d were scaled from the rib blueprint.

LOADING

    The flap is subjected to a shock wave impinging side-on to the skin surface. The shock wave results from a ground explosion of 109 Mg (120 tons) of ANFO at White Sands Missile Range testing grounds at 1220 m (4000 ft) elevation. The flap is located approximately 305 m (1000 ft) from the burst center, and the resulting free-field incident overpressure, p_{so}, is 25.5 kPa (3.7 psi). The pressure loading normal to the panel surface is calculated with the standard method given in Ref.10 and the pressure-time profile is shown in Fig. A-5. The peak reflected overpressure, Δ_{pr}, is calculated to be 57.2 kPa (8.29 psi) and the diffraction time, t_c, is 2.06 ms. Since the peak structural panel responses occur in less than 1.0 ms, the drag loading is of no consequence in the panel response and the imposed loading could be taken as a step function if desired. The linear decay shown in Fig. A-5 was used in the NOVA program for the panel response calculations.

    The rib experiences loading on its upper and lower surfaces due to the enveloping shock front (the upper surface of the rib faces GZ). For the upper rib surface, the overpressure loading, Δ_p^u, is
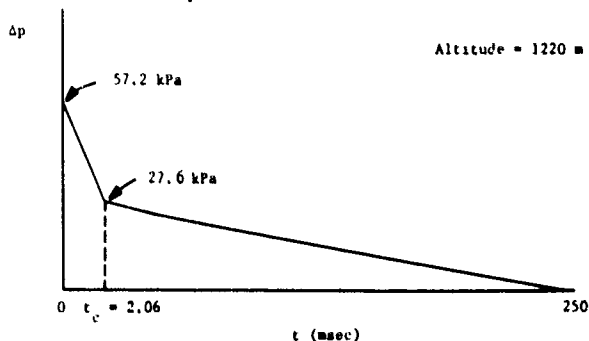
FIGURE A-5.   Pressure Loading Normal to Panel Surface.

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$$P_{so} = 25.5 \text{ kPa} \qquad \text{Incident} \qquad C_o = 335 \text{ m/sec}$$

$$P_{so} = 87.6 \text{ kPa} \qquad \text{Ambient}$$

$$\Delta_{pr} = 2\ P_{so} \left[\frac{7 p_o + 4 p_{so}}{7 p_o + p_{so}} \right] = 57.2 \text{ kPa}$$

$$t_c = \frac{3h}{C_{ref}} = 2.06 \text{ msec} \quad \text{where}$$

$$C_{ref} = \frac{C_o}{\sqrt{7}} \sqrt{\frac{16(p_{so}/p_o)^2 + 70(p_{so}/p_o) + 49}{6(p_{so}/p_o) + 7}} = 360 \text{ m/sec}$$

$$h = 495/2 = 247.5 \text{ mm}$$

$$\Delta\overset{u}{p} = [\Delta p_r - \Delta p(t_c)]\left(1 - \frac{t}{t_c}\right) + \Delta\overset{u}{p}(t_c) \qquad 0 \le t \le t_c$$

$$\Delta\overset{u}{p} = p_s(t) + C_D^u\, q(t) \qquad\qquad t \ge t_c$$

where

$$\Delta\overset{u}{p}(t_c) = p_s(t_c) + C_D^u\, q(t_c)$$

and

C_D is the drag coefficient

q is the dynamic pressure

For the lower rib surface, the overpressure loading, Δ_p^ℓ, is

$$\Delta\ _p^\ell = \Delta\overset{\ell}{p}(t_c)\ \frac{t}{t_c} \qquad\qquad 0 \le t \le t_c$$

$$\Delta\overset{\ell}{p}(t_c) = p_s(t_c) + C_D^\ell\, q(t_c) \qquad\qquad t \ge t_c$$

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where

$$\Delta \hat{p}^{\ell}(t_c) = p_a(t_c) + C_D^{\ell} q(t_c)$$

For the present application with the NOVA code, the average of the upper and lower pressure loadings was applied equally to the upper and lower surfaces of the rib. Thus, for each surface, the applied (compressive) pressure loading on the rib was taken to be

$$\frac{\Delta \hat{p}^u + \Delta \hat{p}^{\ell}}{2} = \frac{1}{2} \left\{ \left[\Delta p_r - \Delta \hat{p}^u(t_c) \right] \left(1 - \frac{t}{t_c} \right) + \Delta \hat{p}^{\ell}(t_c) \; t/t_c + \Delta \hat{p}^u(t_c) \right\} 0 \leq t \leq t_c$$

$$\frac{\Delta \hat{p}^u + \Delta \hat{p}^{\ell}}{2} = p_a(t) + 0.3 q(t) \qquad t \geq t_c$$

where, for $p_{so} = 25.5$ kPa (3.7 psi) (see Fig. A-5),

$$\Delta p_r = 57.2 \text{ kPa (8.29 psi)}$$

$$t_c = 2.06 \text{ ms}$$

$$t_o = 250 \text{ ms}$$

and

$$p_a(t) = p_{so} \left(1 - \frac{t}{t_o} \right) e^{-\frac{t}{t_o}}$$

$$q(t) = q_o \left(1 - \frac{t}{t_o} \right) e^{-3.5 \frac{t}{t_o}}$$

$$q_o = p_o \left[\frac{5/14 \; (p_{so}/p_o)^2}{1 + \frac{1}{7} \; (p_{so}/p_o)} \right]$$

$$C_D^u = 1.0$$

$$C_D^{\ell} = -0.40$$

Figure A-6 presents a plot of the pressure loading assumed
for each surface of the rib.

STRUCTURAL RESPONSE

Two structural response subroutines in the NOVA code
may be used for obtaining the nonlinear blast response of
the flap skin panel (Ref. 7). One subroutine is DEPROP,
which treats the panel as a plate structure and is based on
a modal solution. The other is DEPROB, which is a beam
solution and uses a lumped-mass representation. If the
aspect ratio of the panel is sufficiently high, as it is in
the present case, the beam solution will give results which
are comparable to the panel solution at a considerable
saving in computer time. In addition, the beam solution
(DEPROB) is expected to calculate the edge strain more
accurately than the panel solution (DEPROP). The beam
solution uses a unit strip of the panel of length equal to
the panel width to represent the panel response.
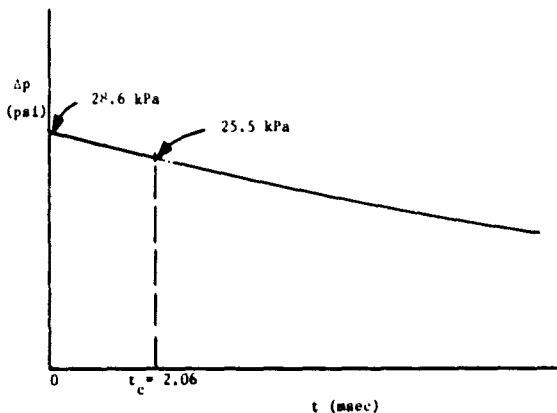
FIGURE A-6.   Pressure Loading Assumed for Upper
and Lower Rib Surfaces.

Both codes are exercised for the present problem, DEPROB for three loading conditions and DEPROP for one loading condition (i.e., the 25.5-kPa (3.7 psi) incident overpressure case). Clamped boundary conditions are used in both codes. The fastener lands introduce an uncertainty regarding the panel dimensions to be used in the codes. Because of this, it was decided to run two cases, one in which the rivet line dimensions are employed and the other in which the rivet line dimensions are reduced by the lands width of 19.1 mm (0.75 in.), in anticipation that the correct model would fall in between.

Figure A-7 summarizes the maximum edge strain and the strain at the panel center as a function of reflected over-pressure, Δp_r. The maximum edge strain occurs in the outer panel surface and the maximum center strain occurs in the inner panel surface. It is seen that at Δp_r equal to 57.2 kPa (8.29 psi) (which corresponds to the 25.5-kPa (3.7 psi) incident level), the maximum edge strain is very close to the yield strain whereas the maximum center strain is considerably lower than yield strain. At the higher pressure levels in Fig. A-7, which extend to 300 percent of Δp_r equal to 57.2-kPa (8.29 psi), the edge strain exceeds 10 percent whereas the center panel strain remains below yield strain. The beam results shown in Fig. A-8 are based on an elastic-plastic solution using 10 masses for a half-beam length, which is allowed by symmetry. The single panel result shown in the figure is based on an elastic solution using a 7 x 7 modal formulation, of which 25 modal combinations are selected.

Some typical strain histories are shown in Figs. A-8 to A-10, in which comparisons are made between the beam and panel solutions for p_{so} of 25.5 kPa (3.7 psi). The two solutions are in very close agreement with respect to peak values and the shape of the response curves. The peak strains and deflections, incidentally, occur at about 0.5 - 0.6 ms, well within the blast diffraction phase.

The rib response is obtained with the beam code DEPROB by selecting a unit strip of the web between the first two lightening holes. The length of the beam strip is 93.7 mm (3.69 in.) and its thickness is 0.81 mm (0.032 in.). Sliding, clamped boundary conditions, characterized by zero slope and only axial motion at the edge, are used. In DEPROB, a
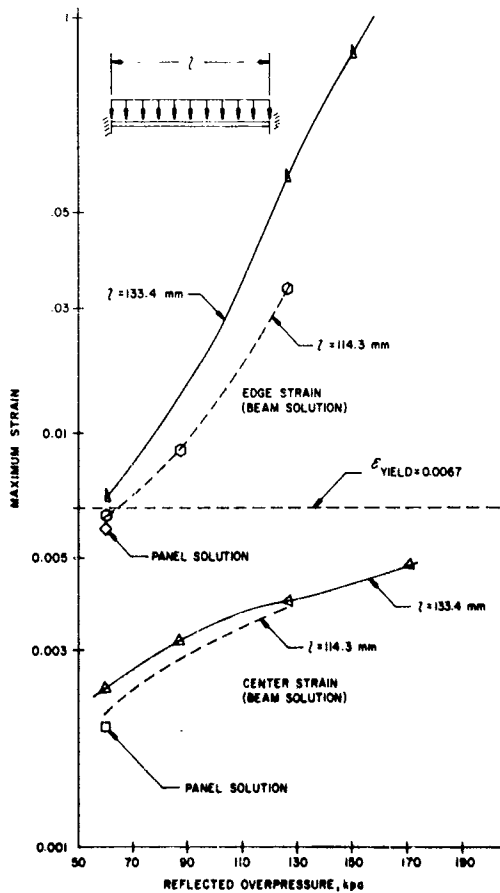
FIGURE A-7. Maximum Strain vs Reflected Overpressure for Skin Panel.
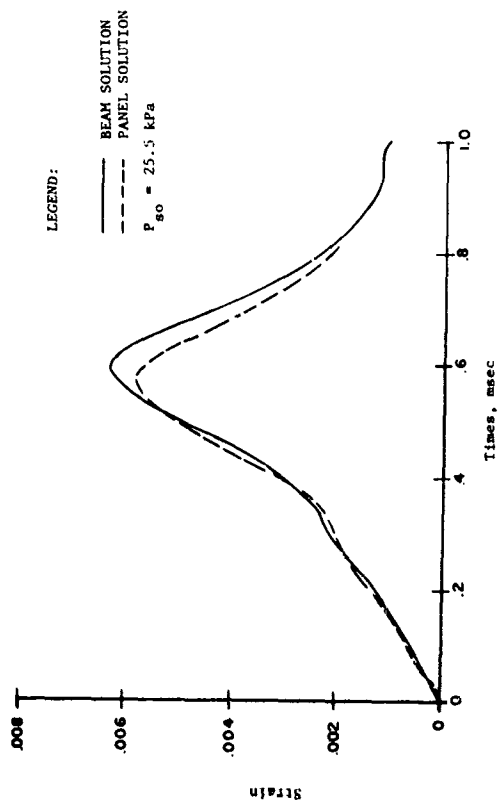
FIGURE A-8. Comparison of Skin Panel Edge Strains, Outer Surface.

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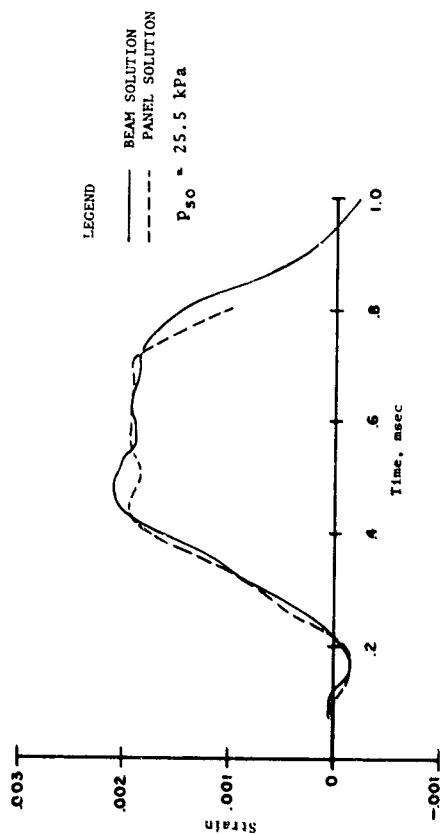
FIGURE A-9. Comparison of Skin Panel Center Strain, Inner Surface.

180
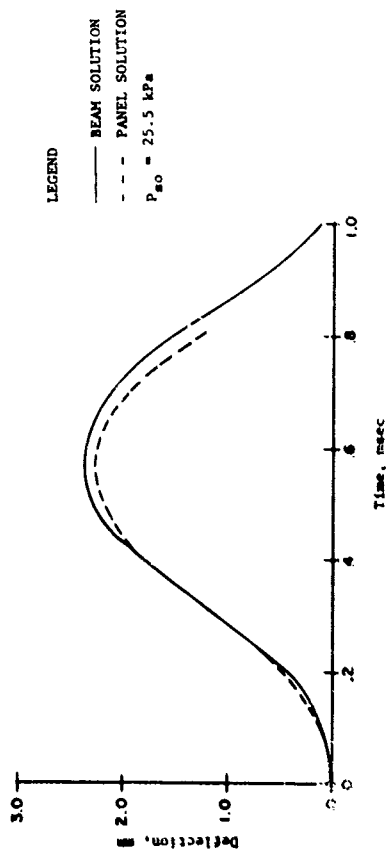
FIGURE A-10. Comparison of Skin Panel Center Deflections.

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$\frac{\delta}{2}\left\{1-\cos\frac{(2\pi x)}{\ell}\right\}$ variation is assumed for the shape of the initial imperfection, where δ is the peak amplitude of the imperfection. Two values of δ are investigated, as δ of 0.254 mm (0.01 in.) and 0.508 mm (0.02 in.). The beam strip is subjected to compressive end loads based on the pressure loading discussed earlier, the panel width, and the dimensions relating to the lightening hole radii and the spacing between lightening holes. Thus, for the pressure loading shown in Fig. A-7, for example, the pressure curve is multiplied by

$$133.4 \frac{\ell}{\ell-r_1-r_2} = 310.39\text{mm} \ (12.221\text{in.})$$

to obtain the compressive end loads acting on the strip.

Figure A-11 presents the maximum edge strain and center strains of the rib as a function of the reflected overpressure. At the reflected overpressure corresponding to the 25.5 kPa (3.7 psi) incident shock, the rib strains are considerably lower than the yield strain. Yield is not experienced until the reflected overpressure is increased by 60 percent (for $\delta = 0.508$ mm (0.02 in.)). It may also be noted that the maximum edge strain and center strain at the outer surface are fairly close for both imperfection values. The peak rib strains occurred between 1 and 2 ms for the 25.5 kPa (3.7 psi) shock and between 3 and 4 ms for the higher pressure levels. As expected, these times are considerably higher than those for the peak skin panel strains.

In summary, the results of Figs. A-7 and A-11 indicate that at incident overpressure of 25.5 kPa (3.7 psi), the skin panel will experience strains comparable to the yield strain along its long edge. Strains will be lower than yield at other points on the panel, even at reflected overpressures considerably greater than the reflected overpressure corresponding to an incident shock of 25.5 kPa (3.7 psi). The rib, on the other hand, will experience strains considerably below yield at the 25.5 kPa (3.7 psi) level, and therefore will not buckle at this load level. At pressure levels approximately 60 percent higher, a possibility of rib buckling exists. The predicted strain data are summarized in Table A-1.
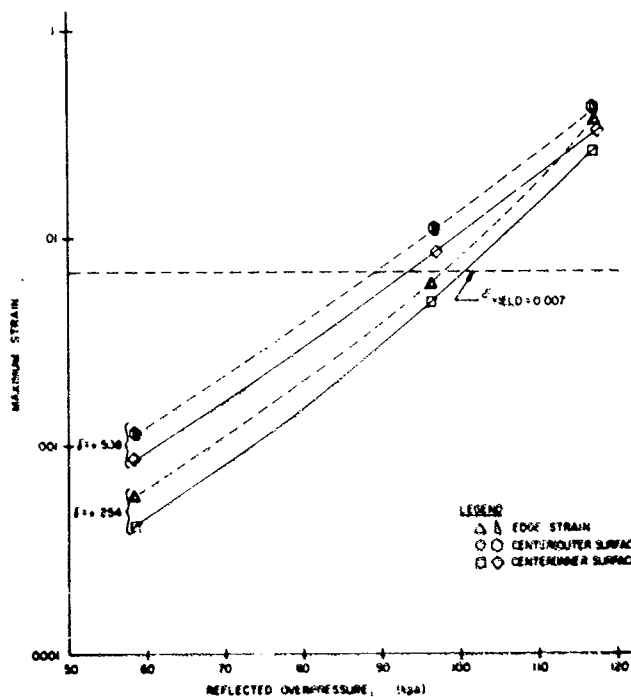
FIGURE A-11. Maximum Strain vs Reflected Overpressure for Rib.

TABLE A-1. Instrumentation Plan.

Gage Number	Panel Number	Gage Location	Surface	Orientation On Centerline	ϵ_{max} Microstrains
1	1	C	OUT	X	-93
2	1	C	IN	X	-85
3	1	C	OUT	Y	-715
4	1	C	IN	Y	1950
5	1	LS	OUT	Y	6500
6	1	LS	IN	Y	-4200
7	1	SS	OUT	X	1709
8	1	SS	IN	X	-2248
9	2	C	OUT	Y	-715
10	2	LS	OUT	Y	6500
11	-	RIB	-	-	1000
12	-	RIB	-	-	1000

Location: Center (C), Long Side (LS), or Shortside (SS)

Surface: Outside (OUT) or Inside (IN)

Orientation: Parallel to Long Side (X) or to Short Side (Y)

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APPENDIX B

EXPERIMENTAL RESULTS OF F-4 TRAILING EDGE FLAP
IN PRE-DICE THROW II EVENT

TEST OBJECTIVE

The NWEF test, conducted as one project in the PRE-DICE
THROW II Event, involved an instrumented F-4 wing trailing
edge flap.

The test objective was to obtain preliminary informa-
tion on structural response to blast. The data from this
test was used in planning the configuration and the instru-
mentation requirements for the A-4C aircraft test specimens
which were subsequently exposed to blast during the main
DICE THROW Event.

TEST SPECIMEN

An F-4 trailing edge flap, shown in Fig. B-1, repre-
sents a structural panel with relatively rigid boundary
members. The aft section was made of honeycomb structure
and the forward section consisted of top and bottom skins
riveted to a substructure made up of a series of light ribs.
The skins, made of 7075 Aluminum alloy were chem-milled to
1.02 mm (0.4 in.) basic thickness with 1.6 mm (0.063 in.)
thick lands.

TEST INSTRUMENTATION

The basic instrumentation consisted of pressure gages
and strain gages. For the purpose of correlating analysis
and equipment, Panel No. 1 and Panel No. 2 (Fig. B-1),
measuring 495 x 133 mm (19.5 x 5.25 in.) between rivet
lines, were the areas selected for the instrumentation.

Strain gages were installed on the upper skin outside
and inside surface as well as on the web of the rib which
made up the outboard boundary of Panel No. 1. Detailed
disposion of the instrumentation is shown in Figs. B-2 and
B-3 for the outer and inner skin surfaces, respectively.
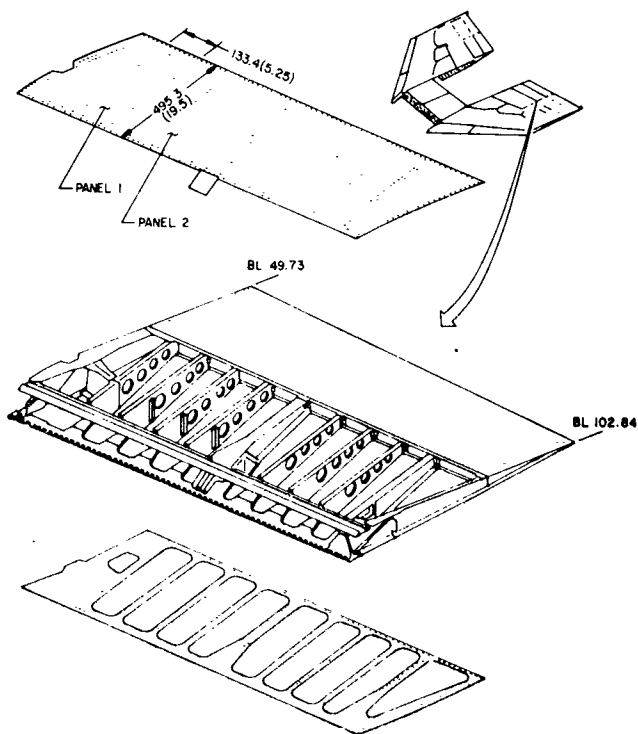
The length is in millimeters
with inches in parenthesis.

FIGURE B-1. F-4 Flap - Structure Configuration Diagram.

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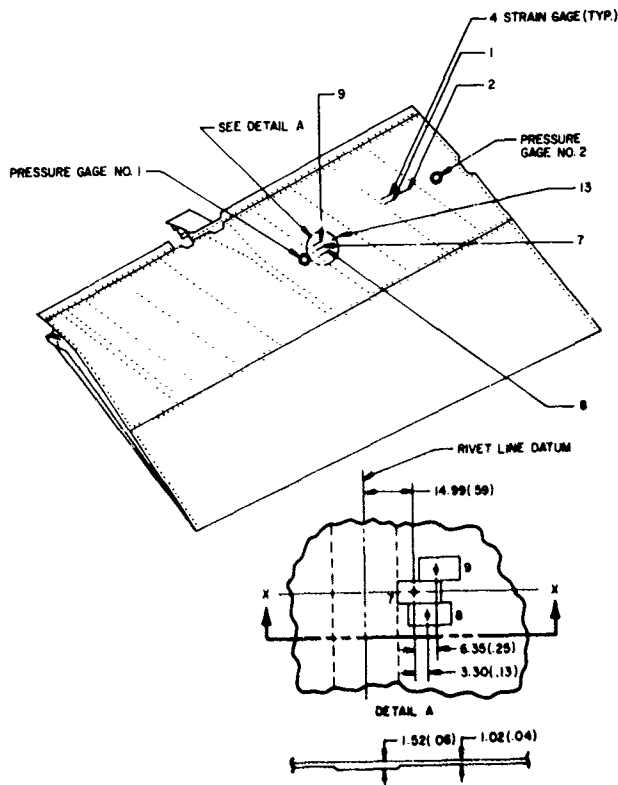
The length is in millimeters
with inches in parenthesis.

FIGURE B-2. Instrumentation Location Diagram · F-4 Flap
Upper Skin Outer Surface.
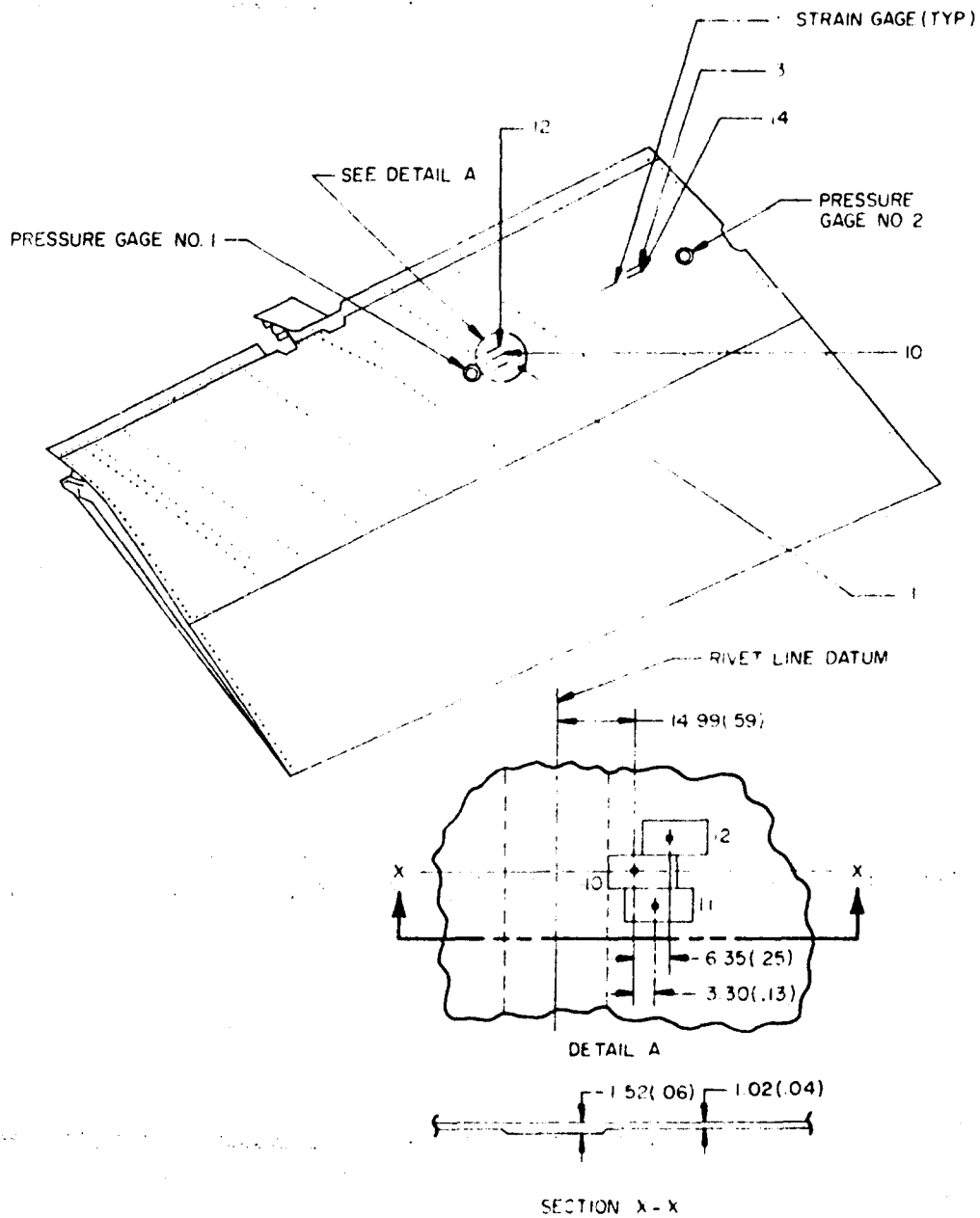
STRAIN GAGE (TYP)

3

4

SEE DETAIL A

PRESSURE
GAGE NO 2

12

PRESSURE GAGE NO. 1

10

RIVET LINE DATUM

14 99(59)

X   X

2

10

11

6 35(25)

3 30(.13)

DETAIL A

1 52(06)   1 02(.04)

SECTION X - X

The length is in millimeters
with inches in parenthesis.

FIGURE B-3.   Instrumentation Location Diagram - F-4 Flap
Upper Skin Inner Surface.

TEST SETUP

The flap panel was mounted vertically on two poles located at a distance of 305 m (1000 ft) from the GZ. See Fig. B-4. Approximately 25.5-kPa (3.7 psi) overpressure was expected at that location. To minimize edge effects which produce local pressure variation, plywood panels extended the flap boundaries in the vicinity of the instrumented areas.

TEST RESULTS

The pressure data obtained from pressure gage No. 1, Fig. B-5, indicated peak overpressure of 57.2 kPa (8.3 psi). The Ballistic Research Laboratory data taken on the master gage line showed a free-field overpressure of approximately 27.6 kPa (4 psi) at the 305 m (1000 ft) radial distance. This yields a reflection factor of 2.1.

The strain gage maximum response data during the first cycle immediately following the blast intercept are summarized in Table B-1. Generally higher strain values of up to 2100 microstrains were obtained at the panel center inside surface as compared to 1680 microstrains maximum indicated for the midpoint of outside surface of the panel longer side. Since the highest strains for a clamped panel were expected at the midpoint of the boundary of the panel longer side, this might suggest that the effective fixity of the panel boundaries was less than rigid. However, it must be noted that, because of the rivet line and substructure support the exact location of the panel boundary is debatable. Consequently, the area along the panel edges where strain gages were installed may only be defined as the vicinity of the panel boundary. Existence of a steep strain gradient at these locations implied some measure of fixity even though the actual strain level at the effective panel boundary remained undetermined.

The presence of steep strain gradients in the vicinity of the panel boundary was convincingly demonstrated by the strain record of gages 7, 8 and 9 as one set and gages 10, 11 and 12 as the second set. These sets of gages indicated strain gradients in excess of 300 microstrains per millimeter.

An interesting observation is implied in the comparison of the strain data obtained from the gages installed along the longer side edges of panels 1 and 2. Panel 1 was bounded

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FIGURE B-4. Test Setup.

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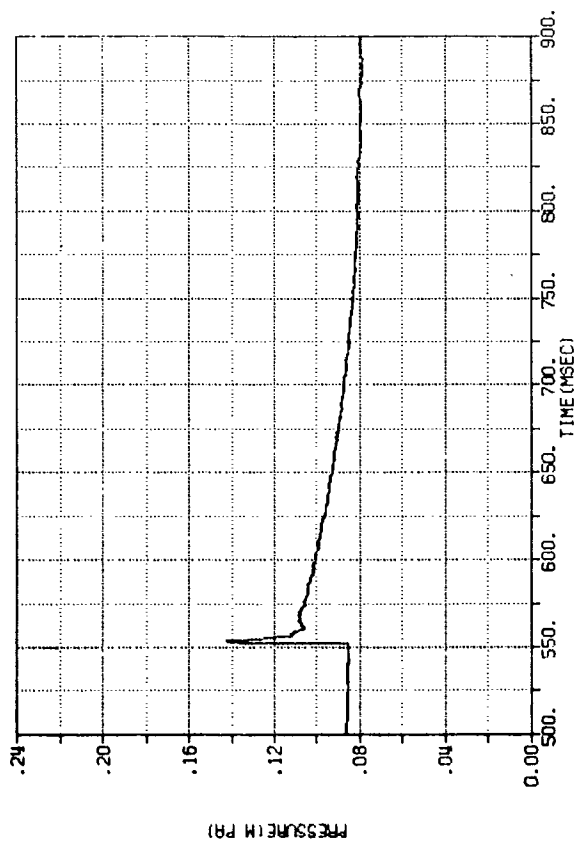
FIGURE B-5. Local Pressure Data - Pressure Gage No. 1
(Gage location shown in Fig. B-2)

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TABLE B-1. Experimental Strain Data,
First Cycle Response.

GAGE LOCATION DATA			GAGE NO.	MAX STRAIN MICROSTRAINS	TIME AFTER SHOCK ARR.
REGION	PANEL NO.	SURFACE			ms
CENTER	1	OUTER	2	-1160	0.68
CENTER	2	OUTER	13	-1180	0.63
CENTER	1	INNER	3	2060	0.68
CENTER	1	INNER	14	2100	0.68
MID LONG SIDE	1	OUTER	1	1280	0.63
"	1	OUTER	4	1300	0.59
"	2	OUTER	7	1680	0.73
"	2	OUTER	8	1220	0.68
"	2	OUTER	9	1150	0.68
MID LONG SIDE	1	INNER	5	-500	0.63
"	2	INNER	10	-570	0.73
"	2	INNER	11	-400	0.68
"	2	INNER	12	-40	0.68
RIB	1	INNER	16	-400	1.19
RIB	1	OUTER	15	2720	1.19

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by a rib of single thickness. There, a strain of 1300 microstrains was obtained. The flanges and the web of the panel 2 boundary rib were twice as thick as the panel 1 rib elements and the corresponding strain exceeded 1900 microstrains. This appears to indicate greater degree of fixity associated with stiffer substructure member at the panel boundary.

The maximum response data presented in Table B-2 in correlation with the first cycle data of Table B-1 show generally higher strain values obtained during subsequent oscillations. This implies a definite relationship between the responses of the individual elemental panels and the flap as a whole. It appears that the maximum response occurred when the elemental panel and the flap bending oscillations were momentarily in phase.

Along the lines of this reasoning, one might anticipate that the differences between the maximum and the first cycle responses should also be dependent on the panel location with respect to the flap boundary. Thus, greater differences in the strain values are expected for panel 2 than panel 1, since the former was farther away from the flap boundary than the latter. This, in fact, was the case as illustrated by the strain data of gages 2 and 13. For the gage 2 located on panel 1 the indicated strain difference was 640 microstrains and for gage 13 on panel 2 that difference was 780 microstrains. This situation is consistely apparent from the panel 1 data (gages 1 and 4) on one hand and the panel 2 data (gages 7, 8 and 9) on the other hand. There, the strain differences averaging to 35 microstrains for panel 1 were contrasted with the average difference of 270 microstrains for panel 2. This behavior pattern focuses on the interdependence of the response of a structural element comprising a part of a larger assembly.

The strain data of gages installed on the panel 2 rib (see Fig. B-6) indicates a maximum strain of 2720 microstrains for gage 15 which was located on the outer face of the rib. The corresponding strain on the rib inner face, obtained from gage 16 was -400 microstrains. These data show the presence of considerable spanwise bending, as illustrated by the differences in the instantaneous strain values. The graph also shows the trace of the direct strain profile. The response cycle of just over 2-ms duration is indicated with the direct strain initially compressive changing to tensile with a maximum value of 2720 microstrains at approximately 1.19 ms after shock arrival.

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TABLE B-2. Experimental Strain Data, Maximum Response.

GAGE LOCATION DATA			GAGE NO.	MAX STRAIN MICROSTRAINS	TIME AFTER SHOCK ARR. ms
REGION	PANEL NO.	SURFACE			
CENTER	1	OUTER	2	-1800	3.5
CENTER	2	OUTER	13	-1960	4.0
CENTER	1	INNER	3	2150	3.5
CENTER	1	INNER	14	2220	3.6
MID LONG SIDE	1	OUTER	1	1350	1.8
"	1	OUTER	4	1300	0.59
"	2	OUTER	7	1940	2.9
"	2	OUTER	8	1500	4.5
"	2	OUTER	9	1420	4.5
MID LONG SIDE	1	INNER	5	-1050	2.2
"	2	INNER	10	-1730	4.5
"	2	INNER	11	-1690	4.3
"	2	INNER	12	-1330	4.3
RIB	1	INNER	16	-400	1.19
RIB	1	OUTER	15	2720	1.19

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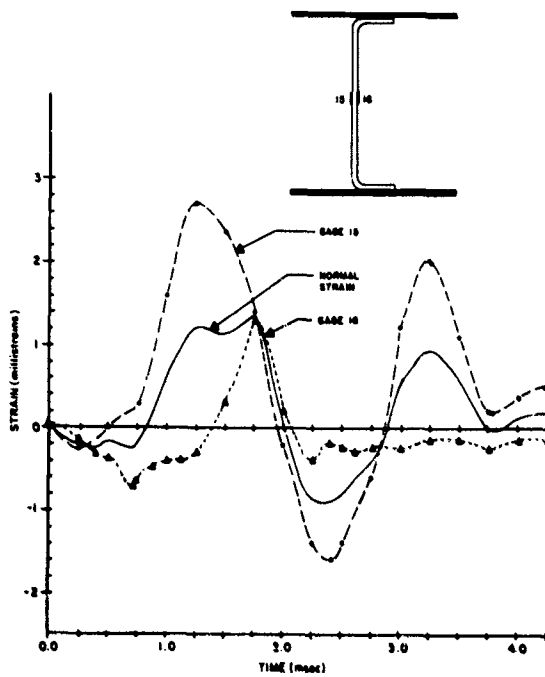
FIGURE B-6. Rib Gages Response.

DATA CORRELATION

The analytic prediction of the panel response to blast was provided by Kaman AviDyne of Burlington, MA. (See Appendix A.) The data was obtained from a NOVA 2 code solution.

As shown in Table B-3 a reasonable agreement exists between the analytic and the experimental data. At the center of the panel inside surface, the analytic strain of 1950 microstrains is only slightly smaller than the corresponding experimental strain of 2100 microstrains. The discrepancy between the strain data for the outside surface (-715 for the analysis was opposed to -1180 and -1160 microstrains for the experiment) may be attributed to somewhat higher membrane stresses predicted in the analysis, whereas a higher degree of bending was indicated by the experimental data (gages 2 and 3).

Considerable discrepancy exists between the analytic and experimental strains at the midpoint of the panel longer edges. The analysis predicts strain level approaching yield condition (6500 microstrains) whereas the experimental strains were less than 1700 microstrains.

As already mentioned, the panel boundary could not be precisely defined. Nevertheless, the high strain gradient indicated for this region tends to support the analytic prediction of high strain level, possibly approaching yield at the panel boundary.

A significant difference between the predicted and the actual strain data is also indicated for the rib response: 1000 as opposed to 2720 microstrains, respectively. However, closer examination of the experimental data shows considerable bending of the rib web, which was not predicted in the analysis. The maximum direct strain obtained for the rib was assessed at 1380 microstrains, which was reasonably close to the predicted value.

A survey of the strain gage data presented in Table B-1 to determine the time after the shock arrival for maximum response during the first cycle indicated a range of 0.59 to 0.73 ms for the panels. This compares very favorably with the analytic prediction of 0.5 to 0.6 ms. Close correlation between the experimental and analytic data for the maximum first-cycle response time was also indicated for the rib with 1.19 ms and 1.0 to 2.0 ms obtained from the test and analysis, respectively.

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TABLE B-3. Strain Data Correlation.

LOCATION		STRAINS MICROSTRAINS		
Element	Surface	Analysis	Test	Gage No. Ref
Panel	INSIDE	1950	2100 2060	14 3
Center	OUTSIDE	-715	-1180 -1160	13 2
Panel No. 1	INSIDE	-4200	-500	5
Mid Long Side	OUTSIDE	6500	1300 1280	4 1
Panel No. 2	INSIDE	-4200	-570	10
Mid Long Side	OUTSIDE	6500	1680	7
Rib	INSIDE	1000	-400	16
Web Center	OUTSIDE	1000	2720	15

It should be noted that, for the analysis, each instru-
mented panel was assumed as a separate structure. In actual
fact, however, these panels constituted elements of the
multi-panel flap structure. The consequence of this limita-
tion in the analytic modeling is reflected in the overall
maximum response data presented in Table B-2 which shows
consistently higher strain values than the corresponding
data of Table B-1. These strain maxima reached, on the
average, at 3 ms after the shock arrival imply a resultant,
composite response of the elemental panels and the entire
flap structure.

In the overall assessment, this test was considered
extremely valuable. It generated some valid data on struc-
ture response to blast and furnished information required to
specify disposition of the A-4C aircraft specimens within
the blast field for the DICE THROW event. The correlation
of the experimental and analytic data provided an initial
insight regarding the effects of structural modeling and
accuracy of the analytic solutions. The discrepancies
between the analytic and experimental data could be attributed
either to the sensitivity of the analytic solution to given
parameters or to the inherent differences between a "true-
to-life" structure and its idealized representation in the
analysis.

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2º. BLAST DISPLACEMENT IN FIELD FORTIFICATIONS

by

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BLAST DISPLACEMENT IN FIELD FORTIFICATIONS

FOREWORD

INTRODUCTION

Objectives

The objectives of this project were (1) to determine
the displacement-time histories of dummies inside 3- x 6-ft
fighting bunkers and inside a 14- x 14-ft underground per-
sonnel shelter and (2) to use these data to confirm a method
for predicting whole-body translation of personnel in open
structures.

Background

Airblast-displacement effects on personnel inside
field fortifications have received little attention. Be-
cause of the lack of information in this area, blast-casualty
criteria have been based solely on damage to the structure,

and safety and risk criteria have been linked to damage to the fortification or to direct-overpressure effects (Reference 1). Results from a previous field test (Reference 2) have suggested that impact injury associated with whole-body displacement induced by the entering jet flow can occur at overpressures well below those required for injury from direct-overpressure effects or structural collapse.

Laboratory studies of jet phenomena in scale models of field fortifications in a shock tube have resulted in the development of a method for predicting whole-body translation of personnel in open structures. It was desirable to confirm this method using full-scale structures on the Dice-Throw field test.

PROCEDURES

Layout

Three 3- x 6-ft fighting bunkers and one 14- x 14-ft underground personnel shelter were located on the test site. Two of the fighting bunkers were face-on to the charge, one each at the 680- and 820-ft ground ranges, and one bunker was side-on at 820-ft (Figure 1). The predicted peak overpressures at these ranges were 25 and 15 psi, respectively. The personnel shelter was located at the 740-ft range at a predicted overpressure level of 20 psi.

Fighting Bunkers

The geometry and dimensions of the bunkers are given in Figure 2 along with the locations of the dummies, pressure gages, and camera. These bunkers were a modification of the fighting bunker with overhead cover described in Reference 3. They were constructed of 1/8-inch sheet steel welded inside a frame of 2-inch angle iron. The bunkers were placed in excavations and covered with earth and sandbags. An asphalt pad extended 70 ft toward ground zero in order to reduce the amount of dust carried by the blast wave.

The volume of the bunker was 140 ft^3 and the areas of the firing port and rear entrance were 4.5 and 5.4 ft^2, respectively. Two dummies were placed in each bunker; the one nearer to the camera was kneeling on the firing step and the other was standing on the floor. The heads of the

-2-
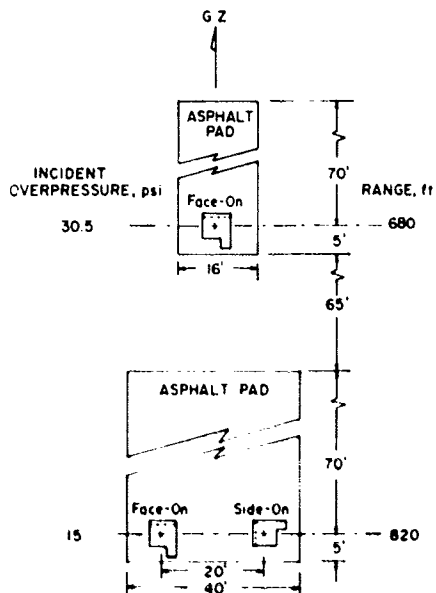
Figure 1. Field Layout of Fighting Bunkers.

-3-
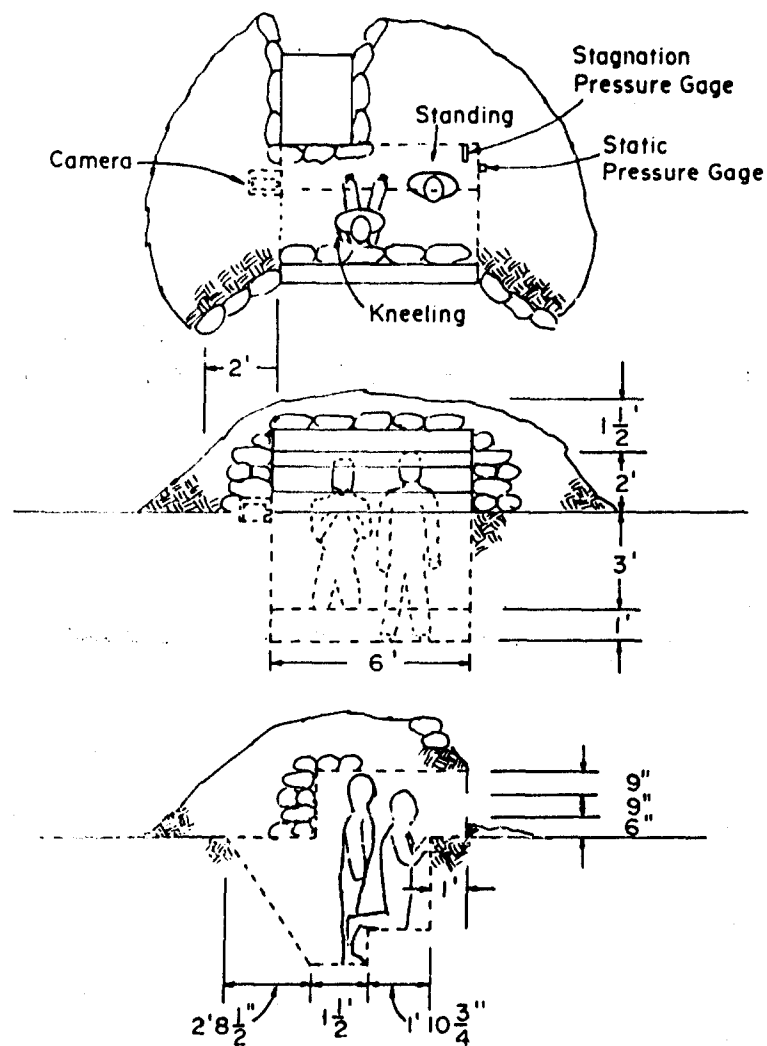
Figure 2.   Diagram of Fighting Bunker Showing Locations
of Dummies, Camera, and Gages.

kneeling and standing dummies were 29 and 12 inches, respectively, from the rear wall. The dummies wore green fatigues, white helmets, and G.I. boots. In Figure 3, the dummies can be seen through the firing port of one bunker. The other two bunkers can be seen in the background.

Personnel Shelter

The geometry and dimensions of the personnel shelter appear in Figure 4. This 14- x 14- x 6.5-ft shelter was identical to the one on the Mixed Company Event (Reference 2) except that the roof was made of steel instead of the 18-inch-diameter pine logs that were used before. An 18-inch I-beam served as the main roof support, and 7-inch I-beams on 2-ft centers spanned between the girder and the side walls. The walls, ceiling, and floor of the shelter were made of 1/8-inch steel plate that was spot welded to the frame. The surface entrance was a 2- x 4-ft opening that was flush with the ground on the upstream side of the shelter. An 8.5-ft-deep vertical shaft and a 6-ft-long tunnel led from the surface entrance to the chamber. The ratio of the chamber volume to the entrance area was 160 ft.

The locations of the dummies, pressure gages, and camera in the personnel shelter are shown in Figure 4. In order to provide photographic reference points, the floor was painted a black and white checkerboard pattern of 1-ft squares and the wall opposite the camera was covered with checkerboard wallpaper. Golden Bear® was applied to the ground surface around and upstream of the entrance in order to reduce the amount of dust carried into the shelter by the blast wave.

Figure 5 is a preshot view of the three dummies which were standing and facing the front wall inside the personnel shelter. Dummy No. 14 was 3 ft from and directly in line with the entryway tunnel. The other two dummies, Nos. 12 and 13, were 3 ft to the side of the centerline extending from the tunnel (Figure 4). The dummies were stabilized in an upright position by leaning them forward at a slight angle against supports made of 1/4-inch pipe anchored to the ceiling.

Photography

The motion-picture photography in the bunkers and shelter was the responsibility of the Denver Research

-5-

Figure 3. Preshot View of the Dummies in the Fighting Bunker at the 680-Ft Range. The two bunkers at the 820-ft range can be seen in the background.
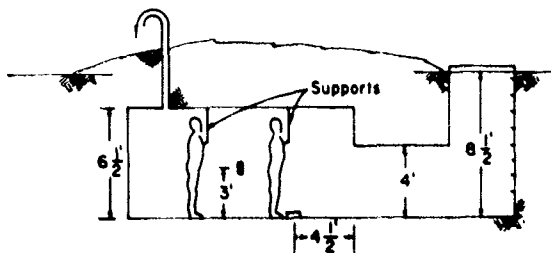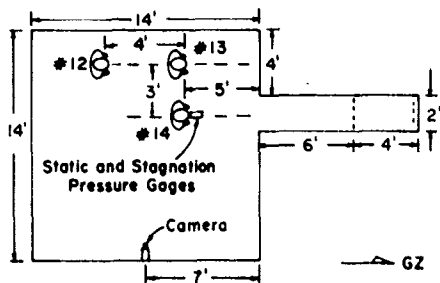
Figure 4. Diagram of Underground Personnel Shelter Showing Locations of Dummies, Camera, and Gages. The surface entrance was at a ground range of 740 ft, 21-psi incident overpressure.
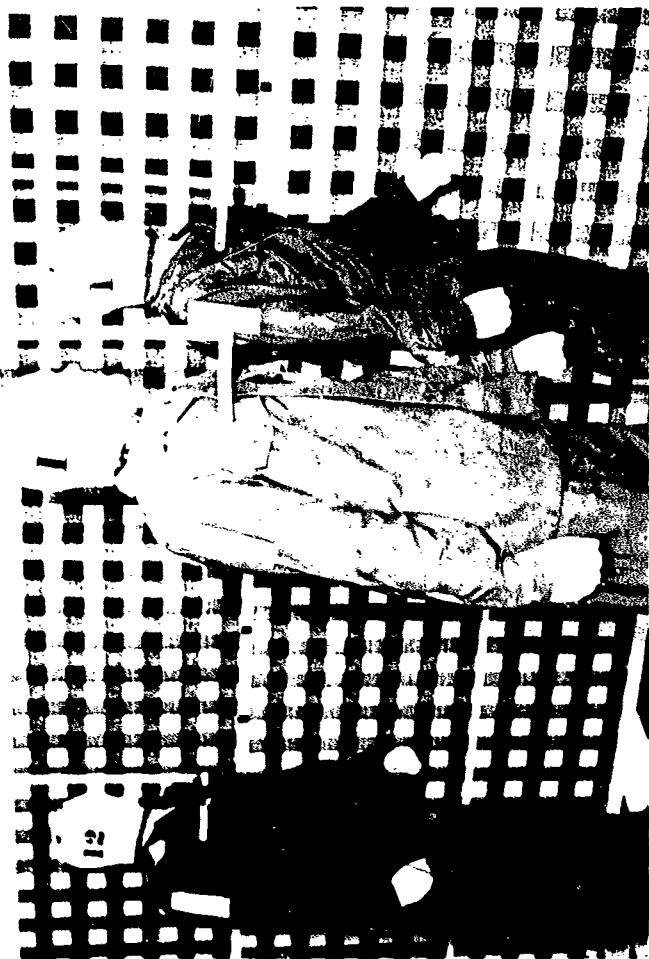
Figure 5. Preshot View Inside Underground Personnel Shelter.

Institute. Each of the four cameras operated at approximately 400 fps.

The pre- and postshot still photography was accomplised by White Sands Missile Range photographers.

Accelercmeters

Four omnidirectional, peak-g, mechanical accelerometers were mounted in the chest cavity of each dummy. Each of these Impact-O-Graphs® contained four small spheres which were pressed into pairs of oppositely facing seats by two interposed springs. The four accelerometers used in each dummy were designed such that the spheres would unload at a peak acceleration of 10, 40, 200, or 800 g's, respectively.

In preshot calibrations, dummies in a prone, supine, or lateral orientation were dropped from various heights onto a concrete pad. For each of these three orientations, the 10-g units unloaded at an impact velocity of approximately 5 ft/sec, the 40-g units at 8 ft/sec, the 200-g units at 17 ft/sec, and the 800-g units at 28 ft/sec. The probabilities of injury for these four impact velocities have been estimated to be 0, 5, 50, and 95 percent, respectively (Reference 4). It should be noted that considerably higher velocities were required to unload the various Impact-O-Graphs® when the dummies were dropped at an angle onto the concrete pad.

Pressure-Time Gages

A static pressure gage was located in each of the three fighting bunkers (Figure 2). In addition, a stagnation pressure gage was located in each of the face-on bunkers in order to measure the jet flow entering through the firing port. The incident free-field pressures were measured by gages on the surface. The gages associated with the bunkers were installed and operated by the Ballistic Research Laboratories.

One static and one stagnation pressure gage were located inside the underground personnel shelter (Figure 4). These gages and one on the surface adjacent to the entrance to the shelter were the responsibility of the Nuclear Weapons Effects Branch of the White Sands Missile Range.

RESULTS

Fighting Bunkers

Dummy Displacements

The postshot locations and conditions of the
dummies in the fighting bunkers are summarized in Table 1.
The four dummies in the face-on bunkers impacted head-first
against the rear wall. This was evident from the postshot
positions and was observed in the motion pictures of the
kneeling dummies. The two dummies in the bunker that was
side-on to the blast had moved approximately 6 inches in
the downstream direction.

Only the 10-g Impact-O-Graph® was unloaded in
each of the four dummies in the face-on bunkers. No Impact-
O-Graphs® unloaded in the two dummies in the side-on bunker.

Only the heads of the kneeling dummies were
visible in the motion-picture films taken in the face-on
bunkers. Figure 6 shows the measured head displacements vs
time. In both cases, dust obscured the initial phase of the
motion. Therefore, those portions of the translation curves
(dashed in Figure 6) were estimated from the initial head
positions and the predicted durations of acceleration.

The peak horizontal component of the head
velocity was 9 ft/sec for the dummy kneeling in the face-on
bunker at an incident overpressure of 15 psi and 24 ft/sec
for the corresponding dummy at 30.5 psi. In each case, the
impact velocity was smaller than the peak velocity. The de-
celeration of the dummy's head prior to impact was probably
related to the fact that the body was rotating. Had the
subject rotated 90 degrees, the horizontal component of the
head velocity would have decreased to the center-of-mass ve-
locity, which was approximately equal to one-half of the peak
head velocity.

The heads of both dummies could be seen in the
film taken in the side-on bunker. The maximum horizontal
component of the head velocity was less than 1 and 2 ft/sec
for the standing and kneeling dummies, respectively (Table
1).

TABLE 1

EFFECTS OF MUNITION IN FIGHTING BUNKERS

Station	Dummy No.	Dummy Pretest Location	Dummy Posttest Location	Condition of Dummy	Film Analysis
Bunker No. 1 - Pretest 58-79 psig 39.8 psi overpressure	3ª	Kneeling	Dummy moved 2 in. to right. Above knees bent, tilted 30 deg backward.	Helmet strap wrapped tightly around neck. Helmet blown off, broken. Shirt blown off, holding on arm only. Dummy retains shoulder of debris impact, girdle on rear wall and face of dummy shows debris impact abrasions.	Dummy rotated backwards head-first, maximum horizontal component of head velocity was 24 ft/sec, velocity decreased to 14 ft/sec prior to impact.
	31ª	Standing	Left supporting arm moved down to side. Dummy fell forward (down left shoulder) against left-hand wall and rotated 70 deg to his right.	Helmet could not be found. Shirt collar torn, both sides along seams. Head impact marks on rear of dummy. Dummy caught on rear wall. Head impact on rear and face of dummy shows debris impact abrasions.	Obscured by dust.
Bunker No. 2 - Pretest 48-79 psig 13.6 psi overpressure	3ª	Crawling	Still in kneeling position tilted 45 deg, head and 45 deg over right arm, rotated forward and over him	Helmet torn from head and neck. Collar torn around both sides along seams. A laceration on back of head. Debris impact marks on face.	Dummy rotated backwards head-first, maximum horizontal component of head velocity was 9 ft/sec, head motion obscured by dust, prior to this displacement at which point the velocity had decreased to 7 ft/sec.
	31ª	Standing	Supporting left arm fell to right 4 in. Legs to original position. Head 4 in. to right.	Helmet scratched at back and twisted to cover left ear. Shirt on neck torn. Debris impact marks on face.	Obscured by dust.

TABLE 1 - CONTINUED

EFFECTS ON DUMMIES IN FIGHTING BUNKERS

Station	Dummy No.	Dummy Preshot Location	Dummy Postshot Location	Condition of Dummy	Film Analysis
Bunker No. 3 - Side-On:					
830-ft range 13.5-psi overpressure	24c	Kneeling	Entire body moved 6 in. to left which is downstream to outside flow.	Helmet in original position but liner shifted downward over eyes. Right side collar torn on side seams. No debris impact.	Dummy rotated backwards head-first; maximum horizontal component of head velocity 1.7 ft/sec; motion obscured by dust after a 6-in. displacement.
	25c	Standing	Entire body moved 6 in. to left. Supporting left arm moved 10 in. to left 5 in. below original position. Body tilted forward.	Right side collar torn on side. No debris impact.	Dummy rotated backwards head-first; maximum horizontal component of head velocity was 0.9 ft/sec; motion obscured by dust after a 2-in. displacement.

Impact-O-Graph® Unloaded:

a Both 10g.

b One 10g.

c None.
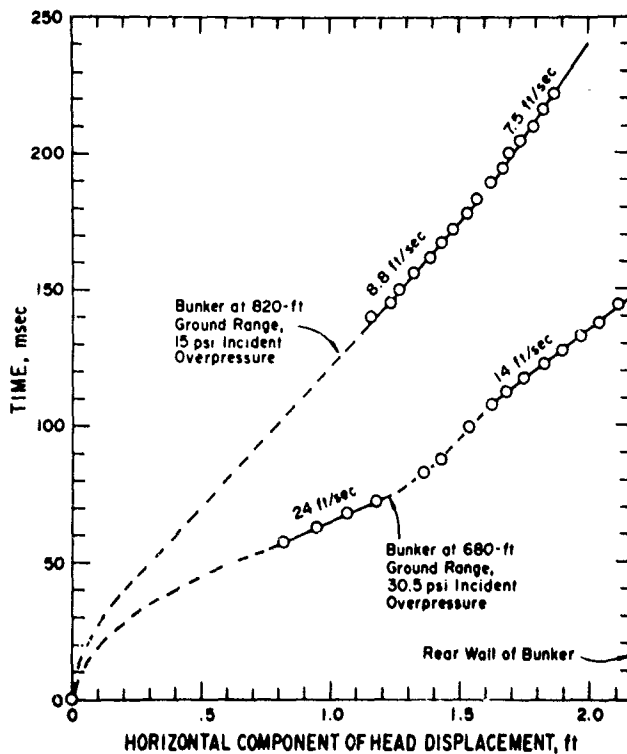
Figure 6. Measured Horizontal Components of the Head
Displacements of the Dummies Kneeling in
the Face-On Bunkers vs Time.

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Pressure-Time Records

Figures 7 and 8 illustrate the smoothed static and Pitot pressures measured in the face-on bunkers, and Figure 9 shows the static pressure measured in the side-on bunker. The prominent pressure fluctuations in these records are believed to be primarily due to compression and rarefaction waves reflecting back and forth within the interior, and therefore do not represent the average pressures throughout the interior or in the jet until the waves are damped out after about 12 msec.

Figures 10 and 11 give the first 15 msec of the free-field pressure-time measurements at the 680- and 820-ft ranges. The peak incident overpressure was 30.5 psi at the 680-ft range and 15 psi at the 820-ft range. Also shown in the figures are the calculated free-field total pressures and the calculated static pressures inside the bunkers, the determination of which will be derived in a following section.

Personnel Shelter

Dummy Displacements

The postshot locations and conditions of the dummies in the underground personnel shelter are summarized in Table 2. Dummy No. 14 (initially in line with the entry-way tunnel) was found against the rear wall of the shelter. Shoe marks on the wall (Figure 12) started 39 inches above the floor, suggesting that the dummy was airborne when its feet first struck the wall. The support pipe broke loose from the ceiling, and Dummy No. 13 fell forward onto the floor with no apparent additional displacement. Dummy No. 12 remained standing, although its shoes had slid backwards about 6 inches. It was not determined if the movements of Dummies Nos. 13 and 12 resulted from airflow or ground shock.

None of the Impact-O-Graphs® unloaded in the three dummies in the personnel shelter.

Because of dust, only the displacement of the head of Dummy No. 14 could be accurately measured from the motion-picture film taken in the personnel shelter. Figure 13 presents the displacement-time measurements of the head of the dummy and the path of its center-of-mass estimated from the predicted acceleration period, the dummy's rotation rate, and its initial and final positions (see Table 2).

-14-
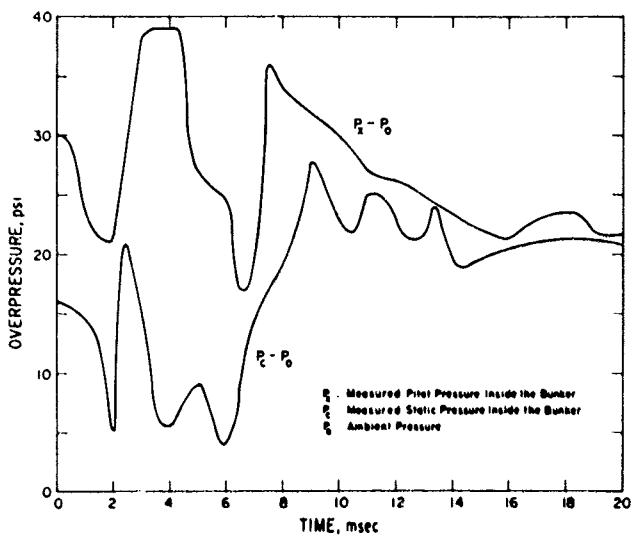
Figure 7. Smoothed Overpressures Measured in the Jet
in the Face-On Fighting Bunker at 680 Ft.
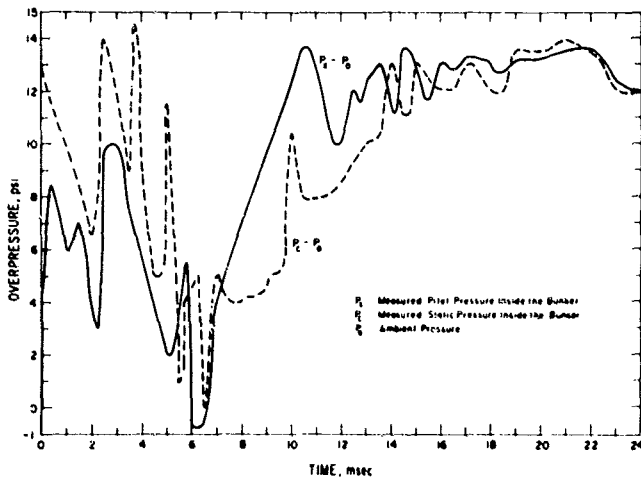
Figure 8. Smoothed Overpressures Measured in the Jet
in the Face-On Fighting Bunker at 820 Ft.
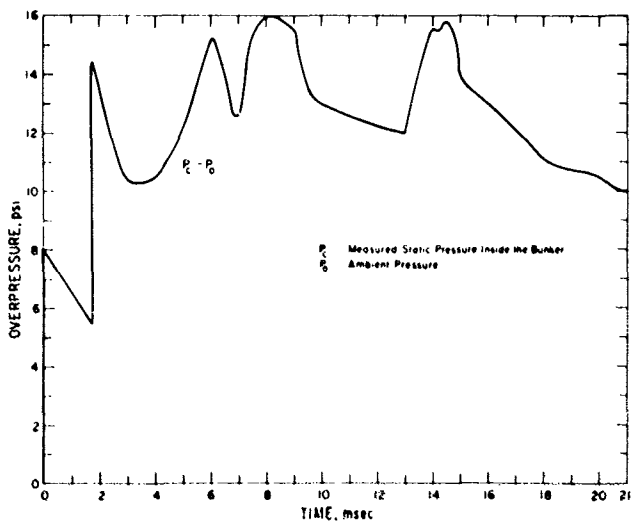
Figure 9. Smoothed Static Overpressure Measured in
the Jet in the Side-On Fighting Bunker at
820 Ft.
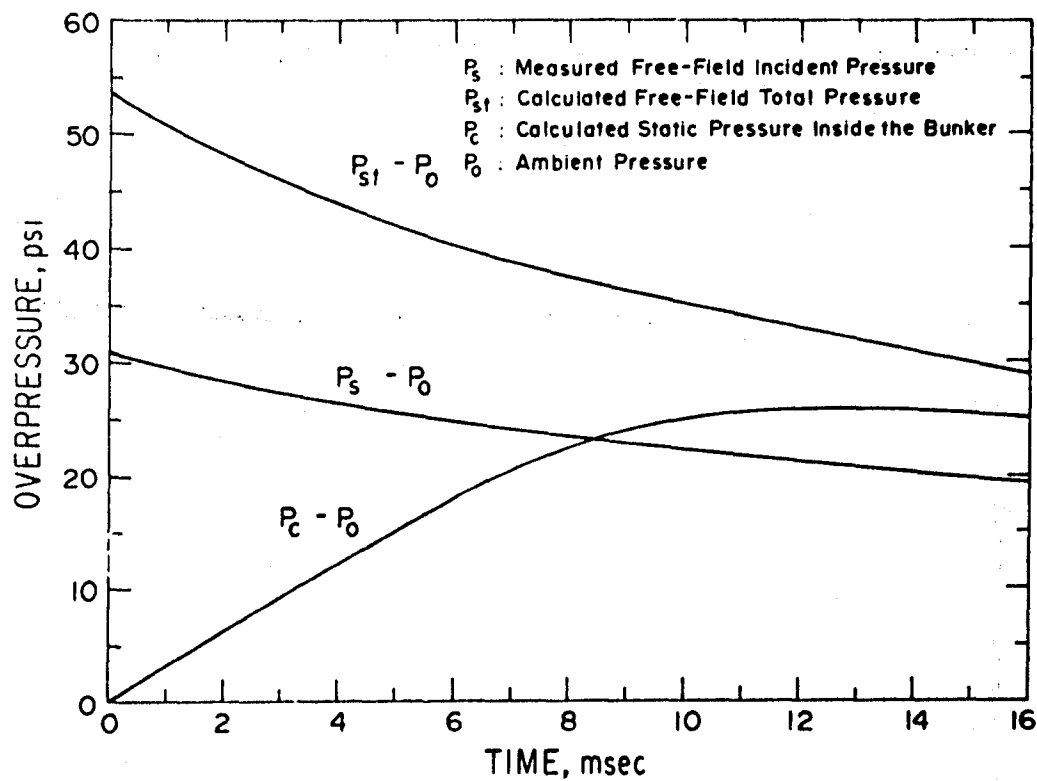
Figure 10. Measured and Calculated Overpressures for the Face-On Fighting Bunker at the 680-Ft Range.

-18-
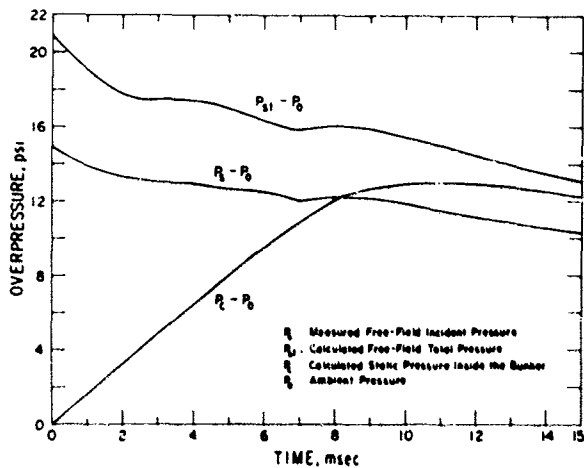
Figure 11. Measured and Calculated Overpressures for the Face-On Fighting Bunker at the 620-Ft Range.

TABLE 2

EFFECTS ON DUMMIES IN PERSONNEL SHELTER

Range, ft	Overpressure, psi	Dummy No.	Dummy Preshot Location	Dummy Postshot Location	Condition of Dummy	Film Analysis
710	21	11*	Standing 5 ft inside and in line with entryway of personnel chamber.	Against rear wall on floor.	Two-in.-long, 1-in.-deep laceration under chin	Dummy moved backwards, rotated feet-first, reversed, impacted at feet-first ~ 14 ft, then impacted floor, head-first ~ 9 ft ...
		13*	Standing 5 ft inside and to the left of entryway.	Fell forward, face down.	No damage.	Obscured by dust
		12*	Standing 10 ft inside and to the left of entryway.	Dummy still standing but shoes had slid backwards 6 in.	No damage.	Obscured by dust.

* Impact-O-Graphs® not unloaded.

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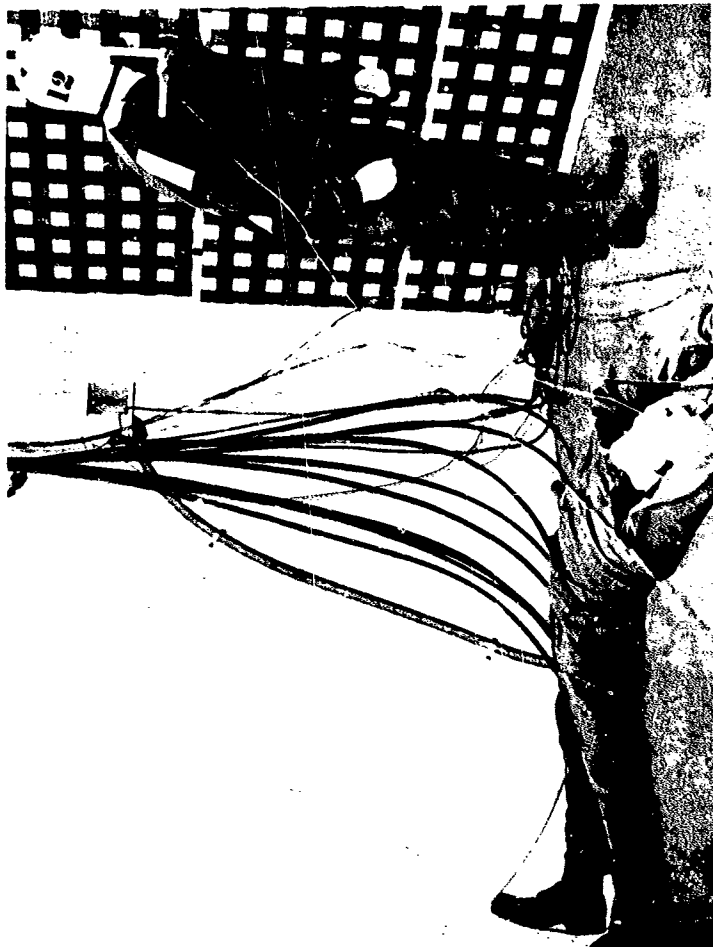
Figure 12. Postshot View of Dummies Nos. 14 (Supine) and 12 (Standing) in the Underground Personnel Shelter. Note the shoe marks on the rear wall.
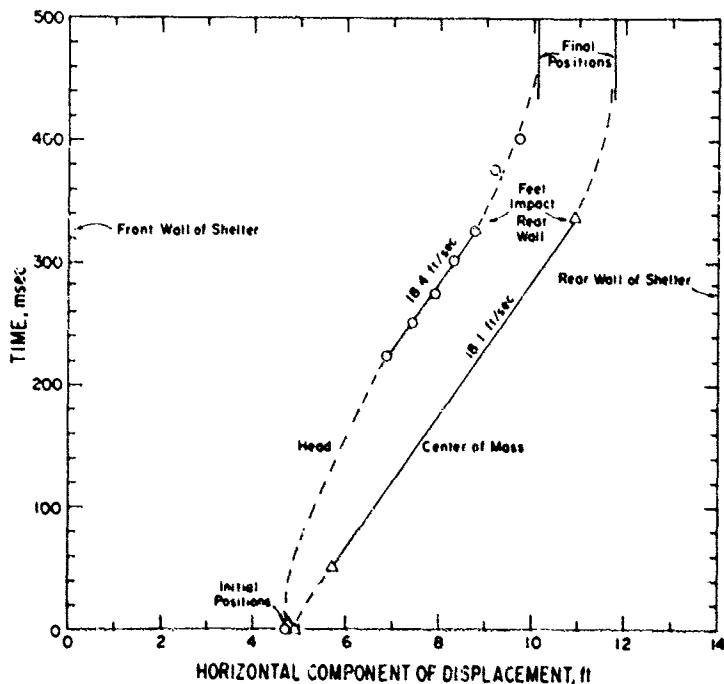
Figure 13. Measured Horizontal Components of the Head and Center-of-Mass Displacements of Dummy No. 14 vs Time. The dummy was standing in the jet entering the underground personnel shelter (see Figure 4).

The horizontal component of the head or center-of-mass
velocity at impact was 18 ft/sec. Prior to impacting the
rear wall, the dummy rotated slightly more than 90 degrees
at a rate of approximately 0.8 rev/sec. It was also de-
termined from the analysis that, shortly after the feet of
Dummy No. 14 struck the rear wall, the head struck the
floor at a vertical component of impact velocity of 9 ft/sec.

Pressure-Time Records

The static and stagnation pressure gages in
the shelter produced poor quality records. Figure 14 shows
the first 60 msec of the free-field pressure-time measure-
ment at the 740-ft range. The measured peak incident over-
pressure was 21 psi. Also shown in the figure is the cal-
culated static pressure inside the shelter.

PREDICTIONS

A method has been developed for predicting whole-body
translation induced by jet flow entering an open structure
subjected to airblast. The procedure involves the sequential
calculation of (1) the external pressures on the openings,
(2) the static pressure inside the structure, (3) the indi-
cated dynamic pressure in the jet, and (4) the resultant ac-
celeration of the dummy exposed to the jet, all of which vary
with time.

External Pressure on an Opening

The external pressure on an opening into a
structure is the driver pressure for the jet entering through
that opening. The external pressure on an opening that is
side- or face-on to the incident shock wave may be assumed to
be the free-field incident or total pressure (i.e., the pres-
sure that arises when the free-field flow is brought to rest
isentropically and adiabatically), respectively. Assuming
that classical conditions describe the free-field shock wave,
the incident and total pressures and the corresponding speeds
of sound are given by the following
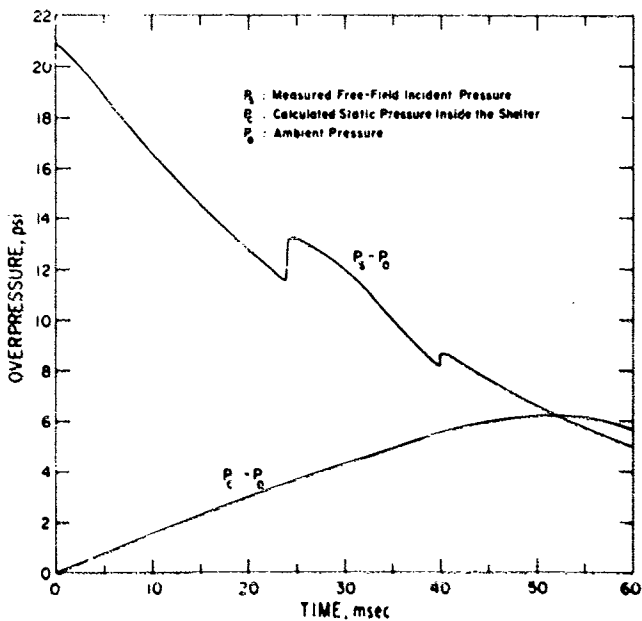
Figure 14. Measured and Calculated Overpressures for the Underground Personnel Shelter at the 740-Ft Range.

Incident pressure ratio

$$P_d/P_o = P_s/P_o$$

Incident sound speed ratio

$$A_d/A_o = A_s/A_o = \left(P_s/P_o \ \frac{P_s/P_o + 6}{6 \ P_s/P_o + 1}\right)^{1/2}.$$

(1)

Total pressure ratio

$$P_d/P_o = P_{st}/P_o = (P_s/P_o)\left[1 + \frac{5 \ (P_s/P_o - 1)^2}{7(P_x/P_o)(P_s/P_o + 6)}\right]^{1/2}$$

Total sound speed ratio

$$A_d/A_o = A_{st}/A_o = (A_s/A_o)(P_{st}/P_s)^{1/7}$$

(2)

where P is pressure and A is the speed of sound, and the subscript o indicates ambient conditions, d indicates ex-ternal conditions at the opening, s indicates free-field shock conditions, and st indicates the conditions that arise when the free-field flow is brought to rest isentropi-cally and adiabatically.

In the case of the fighting bunkers facing the charge, the free-field total pressure, Equation 2, was applied to the firing port and the incident pressure, Equation 1, was applied to the rear entrance. In the case of the personnel shelter and the side-on bunker, the inci-dent pressure was applied to all openings.

Static Pressure Inside a Structure

The jet flow into a structure with one or more openings causes the static pressure in the interior

to increase. The incremental change in the static pressure in the structure is given by

$$\Delta P_c = \Delta t \sum_n (B_n R_n / W) \tag{3}$$

where ΔP_c is the change in the internal static pressure (P_c) during the time interval Δt, W is the volume of the structure, B_n is the area of the nth opening, and R_n is a flow factor for the nth opening. The R's are defined as follows:

For supersonic inflow $P_d > 1.893 \ P_c$

$$R = (175/216)A_d \ P_d \ (P_c/P_d)^{2/7} \tag{4}$$

For subsonic inflow $P_c < P_d < 1.893 \ P_c$

$$R = (7/\sqrt{5}) \ A_d \ P_c \left[1 - (P_c/P_d)^{2/7} \right]^{1/2} \tag{5}$$

For supersonic outflow $P_c > 1.893 \ P_d$

$$R = -(175/216)A_c \ P_c \tag{6}$$

For subsonic outflow $P_d < P_c < 1.893 \ P_d$

$$R = - (7/\sqrt{5}) \ A_c \ P_d \ (P_c/P_d)^{2/7} \left[1 - (P_d/P_c)^{2/7} \right]^{1/2} \tag{7}$$

where A_c is the speed of sound in the structure. Equations 1 through 7 were used to calculate the internal static pressure vs time for the face-on fighting bunkers and the personnel shelter (Figures 10, 11, and 14).

Indicated Dynamic Pressure in a Jet

For a jet flowing into a structure through an opening, the indicated dynamic pressure is the difference between the Pitot pressure, P_X, and the internal static pressure, P_c. The Pitot pressure has a maximum value of P_{Xm} at the opening and decreases approximately linearly with distance, X, from the opening. The indicated dynamic pressure as a function of distance can be calculated from the following formulas:

$$P_x/P_c - 1 = (P_{xm}/P_c - 1)\left(1 - \frac{X/R}{X_m/R}\right) \tag{8}$$

where $P_{xm}/P_c - 1 = P_d/P_c - 1$ for $P_d < 1.893\ P_c$

or $P_{xm}/P_c - 1 = 6\left[(P_d/P_c)^{::} - 1\right]\left[\dfrac{(P_d/P_c)^{:'} - 1}{(35/36)(P_d/P_c)^{:''} - 1}\right]^{:} - 1$ (9)

for $P_d > 1.893\ P_c$

and $X_m/R = \left[0.02905 + 0.02121\ (P_d/P_c - 1)^{**}\right]^{-1}$ (10)

where X_m is the distance from the opening at which the indicated dynamic pressure is zero, and R is the radius of a circular opening. For a noncircular opening with an aspect ratio of less than 3:1, an effective radius $R = (B/\pi)^{1/2}$ can be used without introducing significant error.

Acceleration of a Dummy

In order to calculate the acceleration of a dummy, it is necessary to estimate the projected area and drag coefficient of the portion of the dummy engaged by the jet. It was determined that a standing, face-on, clothed dummy has a height of 68 inches, a projected area of 6.8 ft^2, and a drag coefficient of 1.12, the latter measurement having been obtained in a 6-ft-diameter shock

tube. In the fighting bunkers, the jet entering the firing port was approximately the same height as the port, i.e., 9 inches. It was therefore estimated that the jet intercepted 9/68 of the dummy's height and projected area. In the shelter, it was estimated that one-half of the projected area of Dummy No. 14 was intercepted by the jet. For want of a better approach, the drag coefficient measured for the entire dummy was assumed to apply to the portions of the dummies engaged by the jets in the bunkers and shelter.

If the velocity of an accelerated object remains small compared to the flow velocity in the jet, the indicated dynamic pressure, $P_X - P_C$, may be used to calculate the motion from

$$\Delta V / \Delta t = (SC_D/M) \, (P_X - P_C) \qquad (11)$$

$$\Delta X / \Delta t = V_i + (SC_D/2M) \, (P_X - P_C) \, \Delta t \qquad (12)$$

where S is the projected area of the object engaged by the jet, M is the mass of the object, C_D is the drag coefficient, V_i is the velocity of the object at the beginning of time interval Δt, and ΔX and ΔV are the change in the object's position and velocity, respectively, during Δt.

Fighting Bunkers

Time-displacement histories for the centers-of mass of the dummies in the face-on fighting bunkers were predicted using the methods described above. In each case, the predicted horizontal component of the center-of-mass velocity reached its maximum value within the first 2 inches of displacement, and there was no significant deceleration prior to impact. The measured maximum horizontal components of the head velocities of the dummies kneeling in the face-on fighting bunkers, the corresponding velocities from 1/7-scale shock-tube experiments, and double the calculated peak velocities are given in Table 3. The calculated center-of-mass velocities were doubled in order to make them comparable to the measured head velocities; i.e., initially, the upper portion of each dummy was rotating head-first such that the head velocity was approximately equal to two times the center-of-mass velocity. The predicted and measured

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TABLE 3

MEASURED AND PREDICTED MAXIMUM HORIZONTAL
COMPONENTS OF HEAD VELOCITIES FOR DUMMIES KNEELING
IN FACE-ON FIGHTING BUNKERS

Incident Overpressure, psi	Maximum Horizontal Component of Head Velocity, ft/sec		
	Dice Throw Measurements	Shock-Tube Model Measurements	Predicted Values
12		9	6
15	9		8
20		13	14
30.5	24		24
53			37

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maximum horizontal components of the head velocities (exclud-
ing the shock-tube data) are shown in Figure 15,

For the dummy kneeling in the face-on fighting
bunker at 30.5 psi on Dice Throw, the ratio of the horizontal
component of impact head velocity to the maximum horizontal
component of head velocity was 0.58. In the shock-tube ex-
periments, corresponding ratios of 0.66 and 0.71 were obtained
at incident overpressures of 12 and 20 psi, respectively.
Because the three measured ratios were reasonably close to-
gether, a constant ratio of 0.6 was assumed for making pre-
dictions of impact head velocity for a dummy kneeling in a
fighting bunker vs overpressure. Thus, the curve for a
kneeling dummy in Figure 16 was obtained by multiplying the
predicted velocities from the curve in Figure 15 by a factor
of 0.6.

No velocities were measured for the dummies
standing in the face-on fighting bunkers on Dice Throw.
However, in the face-on fighting bunkers in the shock-tube
experiments, the ratio of the horizontal component of impact
head velocity of a standing dummy to the maximum horizontal
component of head velocity of a kneeling dummy was found to
be 0.84 and 0.87 at incident overpressures of 12 and 20 psi,
respectively. Because the two measured ratios were reason-
ably close together, a constant ratio of 0.85 was assumed
for making predictions of impact head velocity for a dummy
standing in a fighting bunker vs overpressure. Thus, the
curve for a standing dummy in Figure 16 was obtained by
multiplying the predicted velocities from the curve in Fig-
ure 15 by a factor of 0.85.

Data from previous biological studies (Reference
4) were used in connection with the predicted velocities
shown in Figure 16 to obtain the probabilities of impact in-
jury as a function of incident overpressure from a 1-KT yield
for personnel in a face-on fighting bunker, Figure 17.

Personnel Shelters

The predicted horizontal component of the
center-of-mass velocity for the dummy standing in the jet
entering the personnel shelter reached its maximum value
within the first 10 inches of displacement, and there was
no significant deceleration prior to impact. The curve in
Figure 18 shows predicted impact velocity vs incident over-
pressure for a 1-KT yield. The Dice Throw point was plotted

-30-
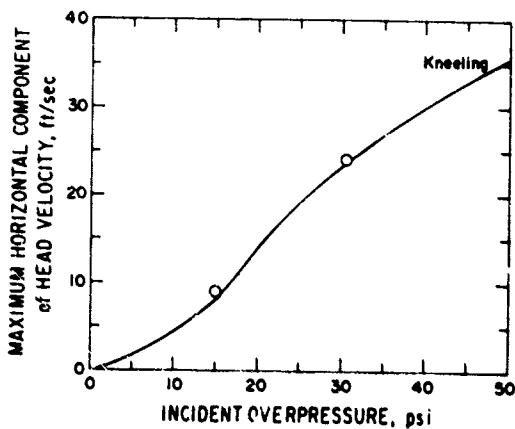
Figure 15. Predicted Maximum Horizontal Component
of the Head Velocity of a Dummy Kneeling
in a Face-On Fighting Bunker vs Incident
Overpressure from a 1-KT Yield. The
points show measured values.
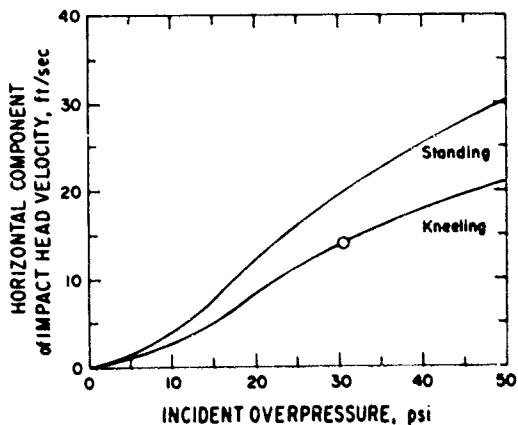
Figure 16. Predicted Horizontal Component of the
Impact Head Velocity of a Dummy Standing
or Kneeling in a Face-On Fighting Bunker
vs Incident Overpressure from a 1-KT Yield.
The point shows measured values.
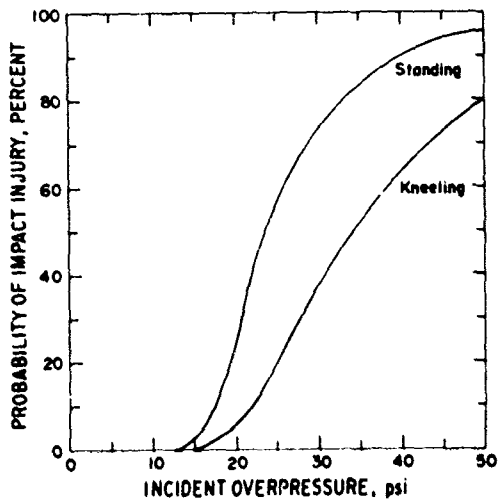
Figure 17. Predicted Probability of Impact Injury to a Person Standing or Kneeling in a Face-On Fighting Bunker vs Incident Overpressure from a 1-KT Yield.

-33-
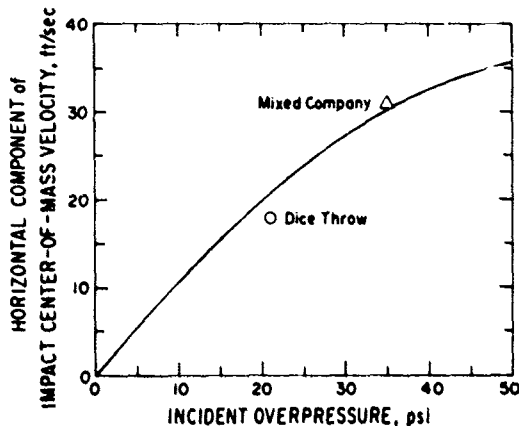
Figure 18.   Predicted Horizontal Component of the
             Impact Center-of-Mass Velocity of a
             Dummy Standing in the Jet Entering an
             Underground Personnel Shelter vs Incident
             Overpressure from a 1-KT Yield. The points
             show measured values except for the Mixed-
             Company velocity which was adjusted as
             described in the text.

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at the measured velocity, whereas the Mixed Company point
was plotted at double the measured velocity. The reason
for doubling the velocity was that (1) the Dice Throw
dummy was standing in line with the center of the entry-
way tunnel such that approximately one-half of its pro-
jected area was engaged by the jet, whereas the Mixed Com-
pany dummy was standing in line with the edge of the entry-
way tunnel such that approximately only one-quarter of its
projected area was engaged by the jet, and (2) the predictions
indicated that the impact velocity should be very nearly
proportional to the projected area engaged by the jet.

Data from previous biological studies (Refer-
ence 4) were used in connection with the predicted veloci-
ties shown in Figure 18 to obtain the probability of impact
injury as a function of incident overpressure from a 1-KT
yield for occupants standing 5 ft from and directly in line
with the entryway tunnel of an underground personnel shelter,
Figure 19.

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     clear Weapons Effects (U)," U. S. Army Combat
     Development Command, Report No. ACN4260, Fort
     Bliss, Texas, 2 August 1971. (Confidential)

2.   Richmond, D. R., E. R. Fletcher, R. K. Jones
     and W. S. Jackson, "Airblast Effects Inside
     Field Fortifications," Mixed Company Event
     Project LN401, POR 6622-1, Defense Nuclear
     Agency, Washington, D. C., 18 June 1974.

3.   "Field Fortifications," Department of the Army
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4.   Fletcher, E. R., J. T. Yelverton, R. A. Hutton
     and D. R. Richmond, "Probability of Injury
     from Airblast Displacement as a Function of
     Yield and Range," Topical Report, DNA 3779T,
     Defense Nuclear Agency, Washington, D. C., 29
     October 1975.
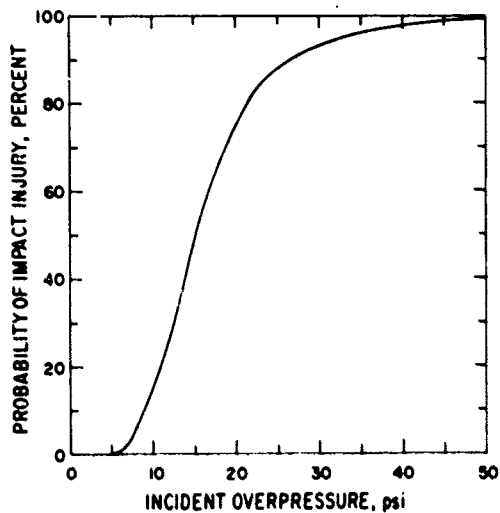
Figure 19.   Predicted Probability of Impact Injury
to Personnel Standing in the Jet Enter-
ing an Underground Personnel Shelter vs
Incident Overpressure from a 1-KT Yield.

30. DYNAMIC RESPONSE OF TWO TYPES
OF GERMAN HOUSE CONSTRUCTION

by

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Stanford Research Institute

CONTENTS

OBJECTIVE

The objective of the SRI program in the Dice Throw event was to in-
vestigate the dynamic response and collapse modes for two types of German
house construction subjected to air blast loading; the two types were
masonry cavity wall and Fachwerk, or half-timber construction. The data
from this experiment is of direct interest to the development of the DNA
collateral damage methodology.

INTRODUCTION

This report is primarily concerned with the post-shot damage survey,
although limited background and construction information is presented for
a better understanding of some of the building damage. Because the elec-
tronic pressure and deflection data are being digitized preparatory to
computer analysis, only preliminary results of the instrumentation is
presented at this time. Also, the high speed films of the test have only
just been received, and although they have been reviewed they have not as
yet been examined in detail.

The detonation occurred at 0800 on October 6, 1976, and re-entry by
the SRI damage survey team was made shortly after the area was cleared,
and the post-shot damage survey of the three test structures was initiated.
The damage survey consisted of a detailed inspection of all masonry walls,
timber members, and debris. Post-shot photographic coverage included the
undisturbed blast damage, as well as photographs of the details of damage
and of reconstructed members considered important for determining the
collapse mechanisms.

It should be noted that the information presented in this event re-
port, including any observations or conclusions, must be considered as
preliminary and are subjected to possible modification when the complete
test data become available.

TEST STRUCTURES

Description

The test structures were approximately 16 ft by 28 ft in plan dimen-
sions and about 20 ft high at the ridge. There were three identical struc-
tures, each located at a different range east of ground zero. As shown
on Figure 1, Structure No. 1 was located at a range of 1140 ft (7.0 psi
predicted free-field overpressure level); Structure No. 2 at 1730 ft (3.5
psi); and Structure No. 3 at 2730 ft (2.0 psi). The front wall of the
structures faced west.

Each structure consisted of two adjoining, but distinct test cells
constructed on a common reinforced concrete slab cast on grade: one
test cell was of masonry cavity wall construction and the other of Fach-
werk, or half-timber construction. The masonry cavity wall test cell

1

STRUCTURE #3
2.0 PSI
2750 FEET
77°-33'-46"

STRUCTURE #2
3.5 PSI
1730 FEET
85°-01'-33"

STRUCTURE #1
7.0 PSI
1140 FEET
91°-45'-34"

BLAST LINE #3

N

GZ
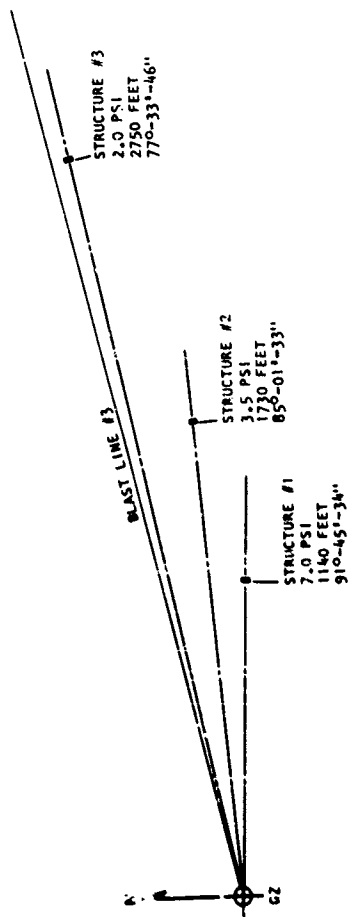
FIGURE 1   LOCATION OF SRI TEST STRUCTURES

2

consisted of unreinforced brick and concrete masonry unit (CMU) wall
panels on the front and one side, and a reinforced concrete ceiling slab.
The Fachwerk wall test cell consisted of brick and timber wall panels on
the front and one side, and a timber joist ceiling system. The back wall
of the test structure and an interior wall separating the two test cells
were constructed of reinforced concrete masonry units (RCMU). The test
structures were covered by a wood frame gable roof system supporting
heavy clay tile roofing. The plan and elevations of the test structures
are shown on Figure 2.

Design

The selection of the walls to be included in the field test was
based on typical types of West German house construction for which no
dynamic response and collapse information was available. One wall type
selected was representative of masonry cavity wall construction found in
load-bearing wall residences of relatively recent periods, and the other
wall type was representative of the traditional Fachwerk (half timber)
construction prevalent throughout Germany during previous periods.

Masonry Cavity Test Wall

The design of the masonry cavity wall test panels was similar to the
exterior walls used in the "Modern Masonry Wall House" described in Ref.
1. However, as a result of differences in U.S. and West German construc-
tion materials, it was necessary to make certain substitutions in the
design of the test walls. For example, the red brick commonly used in
the U.S. has a width of 3-1/2-in. (9 cm) instead of the 11.5 cm (4.5 in.)
prevalent in West Germany. This substitution was felt to be of minor
importance because the interior wythe of CMU and not the exterior facing
wythe of brick was thought to govern the strength of the wall. Of possibly
more importance was the substitution of CMU for the gas concrete blocks
used in the interior wythe of the modern German house in Ref. 1. Because
gas block is manufactured by a special process and is not available in
the U.S., its use in the construction of West German residences was dis-
cussed with an SRI consultant (Ref. 2). It was his opinion that although
gas concrete and other light weight blocks are being widely used in West
Germany today, there are a considerable number of existing buildings with
masonry units similar to the CMU used in the U.S. In any event, for the
specific type of wall tested, the substitution of CMU for gas concrete
blocks was not considered too important because the collapse strength of
the wall is primarily a function of the relatively large in-plane forces
imposed on the wall by the concrete ceiling slab and heavy roof system
rather than of the material properties of the wall itself.

The final design for the masonry cavity test walls is shown on Fig-
ure 2. As can be noted, the overall thickness of the wall was about 14
in., and was constructed with a 3-1/2-in. thick exterior wythe of brick,
a 7-5/8-in. thick interior wythe of CMU, and a 2-7/8-in. wide cavity.

3
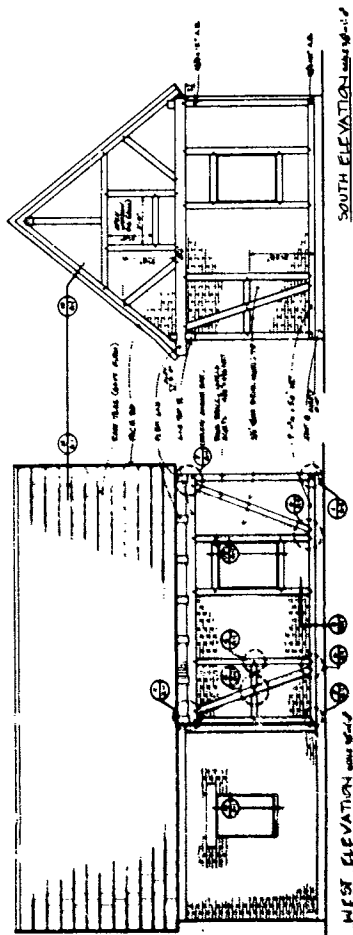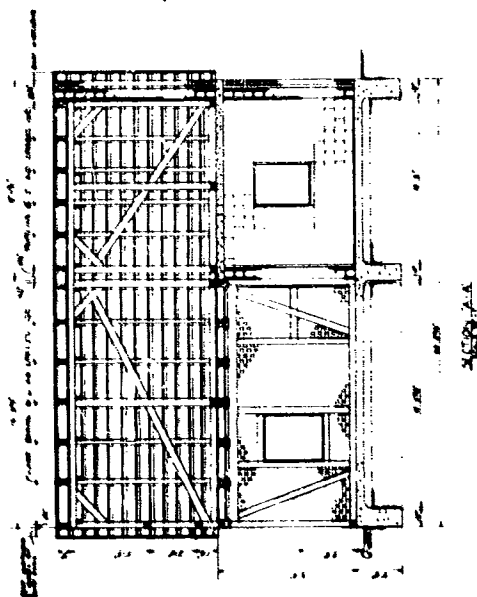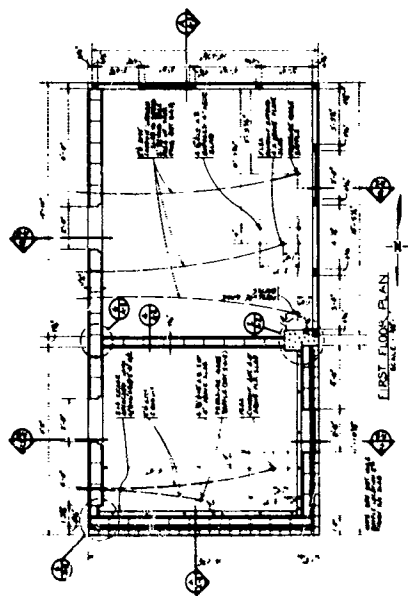
FIGURE 2. PLAN AND ELEVATIONS

Fachwerk Test Wall

The Fachwerk, or half-timber buildings existing in Germany today were built over a period spanning hundreds of years, and consequently were constructed with a variety of sizes and types of timber and masonry materials, and with innumerable patterns or layouts of the timber members. Because it was not possible in a single field test to examine the behavior of all combinations of timber and masonry of interest, a primary goal was to design a test to obtain basic data on the dynamic response and collapse mechanisms of Fachwerk walls that could be used to verify or help develop a prediction model for application to a variety of actual building situations and weapon environments. Also, because of the complexity of Fachwerk construction, it was necessary during the design to include only those details in the test panels that were judged to have significant influence on the response and collapse of the wall under blast loading.

A preliminary design of the test walls was made based on an examination of standard sources of West German construction practices, such as Refs. 3, 4, and 5. The preliminary design was then discussed with the SRI consultant (Ref. 2) and modified to reflect his actual German construction experience. The final design for the Fachwerk test walls is shown on Figure 2. All posts, diagonals, and braces were 4-3/4-in. sq timbers, and all Fachwerk joints were mortise and tenon, connected with wooden pegs. The masonry in-filled panels were 3-1/2-in. thick brick, and had various spans to provide a range of collapse strengths.

The details of the test structures are shown on the construction drawings and specifications (Ref. 6).

Analysis

A dynamic analysis of the two test walls was required to estimate the relative room size of the masonry cavity wall and Fachwerk test cells, and to estimate the collapse overpressure level of the walls for locating the test structures at the site. The ARL predicted air blast data for the DICE THROW event (Ref. 7) was used for all analyses.

Masonry Cavity Wall

The masonry cavity wall was analyzed using the SRI building evaluation procedure for unreinforced masonry walls (Ref. 8). Although the details of the analytic method will not be presented in this event report, the wall was analyzed as a two-way action wall, fixed on four edges, and with vertical in-plane forces resulting from the reinforced concrete ceiling slab and heavy roof system. Because the methodology had not been developed for cavity wall construction, and because it was thought that the two wythes of brick and CMU would act independently as a result of the large cavity, the analysis was based on the resistance of the CMU wall. Under the assumed behavior the brick wall was found to have negligible blast strength.

5

Fachwerk Wall

The collapse analysis of Fachwerk walls required the development of
an interim resistance function and preparation of the computer program,
since this type of wall had not been treated in the previous SRI building
evaluation procedure. To estimate the collapse strength of Fachwerk walls
required an estimate of the response and collapse of at least the two major
components, the in-filled brick panels and the supporting timber posts.

For the analysis of the brick panels it was assumed that a shear fail-
ure between the in-filled brick panels and the timber posts would not oc-
cur during the initial response of the wall because spikes driven into
the timbers and embedded in the mortar between brick courses would prevent
a punching shear failure. Instead, it was assumed for the interim mathe-
matical model that the brick panels would develop an initial bending mode,
and after flexural cracking would respond in a horizontal arching mode
between vertical timber posts. The details of the model are presented in
Ref. 9; however, it was assumed that as the masonry panel deflects under
lateral load, the in-plane movement of the wall at the supports was re-
sisted by the deformation of the vertical timber posts.

A preliminary flexural analysis of the timber posts that support the
brick panels and span between the top and bottom plates, indicated that
the posts would develop a slightly greater resistance than the brick pa-
nels. Also, because mortise and tenon joints were used to connect posts
and plates, a shear failure at the connections was not predicted as would
be the case for ordinary wood stud nailed connections. The pre-test pre-
dictions for the Fachwerk walls was therefore based on the analysis of
the brick in-filled panels.

Collapse Predictions

The SRI pre-shot predictions for the test structures are presented
in Figure 3 for the front and side masonry cavity test walls and Figure
4 for the front and side Fachwerk test walls. As can be noted in the
figures, the results of the collapse analyses are presented as probability
functions. The predicted collapse overpressure is defined as the peak
free-field overpressure that results in the incipient collapse of a wall;
where incipient collapse implies that the wall is on the threshold of
collapse.

For the masonry cavity walls, the predictions on Figure 3 indicate
only a slight chance of the front wall reaching incipient collapse at
the 2.0 psi level, a 0.98 probability of collapse at the 3.5 psi level,
and collapse assured for the front wall at 7.0 psi. For the side wall,
the predictions indicate no chance of collapse at the 2.0 psi level, a
0.50 probability of collapse at the 3.5 psi level, and a 1.0 probability
of collapse at 7.0 psi.

For the Fachwerk test walls, the collapse predictions of Figure 4
show a greater spread than those for the masonry cavity walls; this is

6
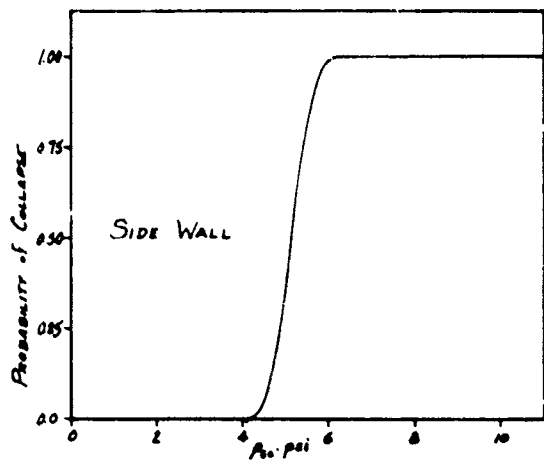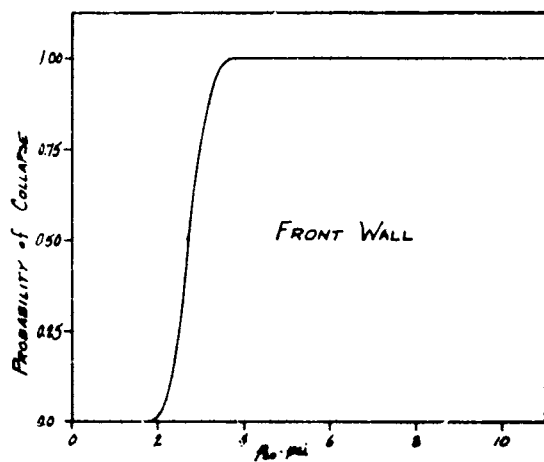
FIGURE 3   COLLAPSE OVERPRESSURE PREDICTIONS FOR MASONRY CAVITY WALL
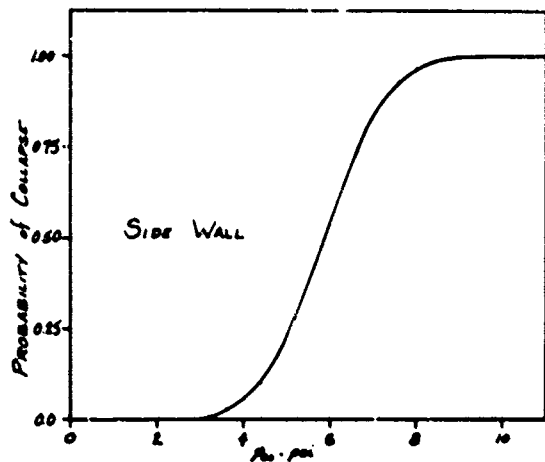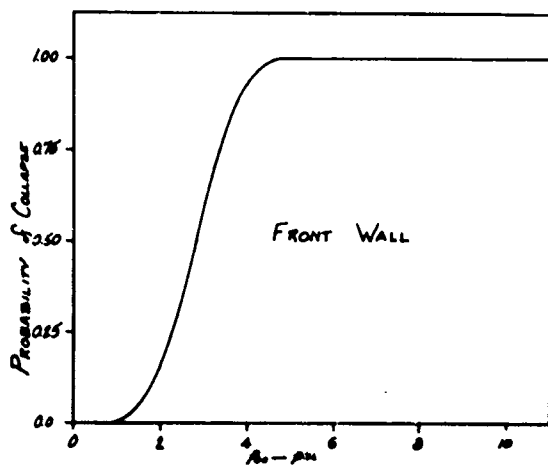
FIGURE 4   COLLAPSE OVERPRESSURE PREDICTIONS FOR 48-IN. WIDE FACHWERK WALL PANEL

N

a result of the unknowns involved in the dynamic analysis of Fachwerk walls. Also, the predictions shown on Figure 4 are only for the 48-1/2-in. wide panel adjacent to the window. For the front Fachwerk wall panel, the predictions indicate a 0.15 probability of collapse at the 2.0 psi level, a 0.80 probability of collapse at 3.5 psi, and collapse assured at the 7.0 psi level. For the side wall, there was no chance of collapse predicted for the 2.0 psi level, only about 0.10 probability of collapse at the 3.5 psi level, and about 0.85 probability of collapse occurring at the 7.0 psi level.

INSTRUMENTATION

The instrumentation for the three test structures included 35 electronic gages, consisting of 26 pressure-time gages and 9 deflection gages, and 6 high speed cameras. Basically the electronic gages consisted of exterior pressure gages to measure the pressure-time function of the front and side test walls; interior pressure gages to measure the pressure transients and the room-pressure build-up for determining the differential pressure-time loading on the test wall panels; pressure gages to measure the pressure-time on the upper and lower surfaces of the second story (ceiling) concrete slab to determine if a significant pressure differential existed on the slab; and deflection gages to measure the dynamic response of selected wall panels. The gage locations are indicated on Figure 5.

All structures were instrumented identically, except that Structure No. 3 (at 2.0 psi) had two additional head-on pressure gages located in the front face of the reinforced concrete pilaster that separates the masonry cavity and Fachwerk test walls. The purpose of the additional pressure gages was to determine if any pressure anomalies occurred on the exterior surface of the front wall as a result of the 1-ft overhang of the second story floor systems.

In addition to the electronic instrumentation, two high-speed cameras were installed at each structure location. As noted on Figures 6a and 6b, the cameras were located north and south of each structure at an angle of 50 degrees from the front wall.

The primary purpose of the high-speed cameras was to record the mode of response and collapse mechanism of each wall element, including individual Fachwerk panels. The cameras were to supplement the electronic gage measurements, and to assist in the post-shot analysis by providing a visual record of the initial and final wall break-up and debris transport.

As noted on Figure 6, the Nova cameras were located at Structure No. 1 and the Photosonic cameras at Structures Nos. 2 and 3. The Nova cameras, which have the shorter focal length lenses, were located at Structure No. 1 to reduce the distance between camera and structure to minimize any dust or debris problem that could occur at the higher overpressure level.

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STR. NO. 3
2 psi
2750'

N

STR. NO. 2
3.5 psi
1730'

STR. NO. 1
7.0 psi
1140'
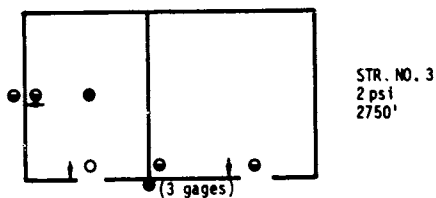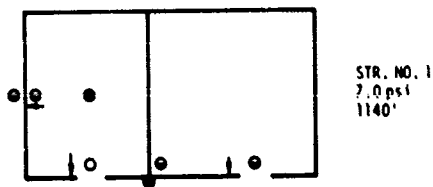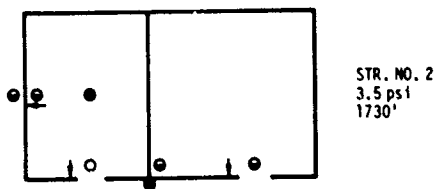
Legend:

   ◐ — Side-on p-t gage, ground slab (or grade)
   ● — Side-on p-t gage, top and bottom of ceiling slab
   ○ — Side-on p-t gage, bottom of ceiling slab
   ● — Head-on p-t gage, concrete pilaster
   ↑ — Deflection-time gage, wall mid-height

FIGURE 5   INSTRUMENTATION PLAN

10
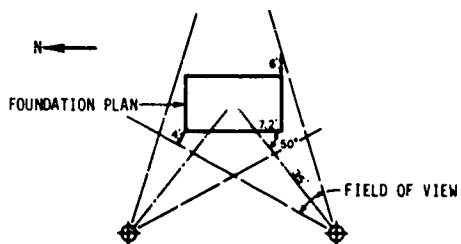
FIGURE 6a   NOVA CAMERA LOCATIONS STRUCTURE NO. 1 (7.0 psi)
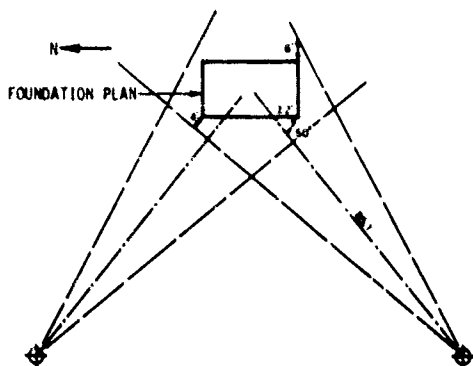
FIGURE 6b   PHOTOSONIC CAMERA LOCATIONS STRUCTURES
NOS 2 & 3 (3.5 and 2.0 psi)

CONSTRUCTION

All three building sites were worked simultaneously although often similar operations were carried out in sequence from the structure nearest to ground zero (G.Z.)(Structure No. 1) to the farthest structure (Structure No. 3).

Foundation and Floor Slab

Construction began with the excavation for the floor slab to an appropriate depth of 6 in. below natural grade; the floor slab, 28 ft 2-5/8 in. by 15 ft 2-5/8 in. in plan dimensions, was oriented with its long dimension perpendicular to the line of site to G.Z. Footings 14 to 24 in. wide were excavated to a depth of 3 ft below top of slab. Soil under the slab was wetted and compacted with gasoline powered, hand-pushed compactors until soil density to a depth of 6 in. was at least equal to natural density. Approximately 6 in. of heavy sand was then spread and compacted on the area below the floor slab and polyethylene sheeting 0.006-in. thick was placed on top of the sand.

After steel forms were fixed in place and reinforcing steel located and tied, approximately 28 cy of ready-mixed concrete, batched on the Dice Throw test bed, was placed and finished. Final slab thickness varied from 6 to 8 in., and the slab was reinforced in both directions by 3/8-in. steel bars 8 in. on center. In Figure 7 the finishing of the slab is in progress. The figure also shows the apportionment of the slab surface into two rooms of unequal areas, separated by a line of reinforcing bars projecting above the slab. These bars, as well as those in the rear wall shown in the right of the photograph, were grouted into concrete masonry units making up the non-responding portions of the structure.

The slab in the foreground of Figure 7 contains the two Fachwerk test walls on the west and south sides; the cavity walls are in the north room beyond the interior divider wall.

Non-Responding Walls

Heavily reinforced to their full height and solidly grouted (except for the divider wall at Structure No. 1), the RCMU walls were designed to resist the dynamic forces without inelastic response. The interior divider wall contained 8-in. thick RCMU; the rear or eastern wall, 12-in. thick RCMU. Both RCMU walls were tied at their intersection by horizontal reinforcing steel. The rear wall had an access opening 3 ft wide by 4 ft 8in. high leading into each room; during the shot these openings were closed with heavy wooden doors bolted in place.

The western anchorage for the interior divider wall was a reinforced concrete pilaster approximately 15 by 27 in. in cross section extending the full height of the wall and tied to it by horizontal reinforcing steel.

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FIGURE 7   FINISHING REINFORCED CONCRETE FLOOR SLAB

In Figure 8 construction of the RCMU walls is underway. Some of the reinforcing steel for the pilaster projects from the slab at the left end of the divider wall.

Masonry Cavity Wall

After the RCMU walls were laid up to a height of about 4 ft, the masons immediately began work on the first story masonry cavity test walls. Like the RCMU the two wythes of the masonry cavity walls were mortared directly to the floor slab, but no reinforcement was used in the test walls. The cavity walls were anchored at the northeast corner of the RCMU wall by an interlocking CMU bond with the RCMU and by steel masonry ties between brick and CMU. At the pilaster, anchorage was achieved through metal ties and a single column of 8-in. RCMU. (See detailed drawings in Ref. 6.)

As the cavity wall reached the 4-ft hieght of the RCMU walls, the masons set up scaffclding and continued construction of both the RCMU and the cavity walls. In Figure 9 the cavity walls are seen nearing completion. The Z-ties between the CMU and brick wythes are in place ready for placement of mortar; the anchorage to the RCMU at the northeast corner is seen in the center foreground.

To simulate a substantial floor system in preventing inward sliding of the cavity wall over the floor slab, a nominally 2-by-4-in. wood plate was bolted to the slab directly against the inside face of the cavity wall. In Figure 7 the bolts for this plate may be clearly seen projecting above the slab on a line about 15-1/2 in. in from the edge.

Aboveground Concrete Work

When the masons had finished all first story masonry walls, the pilaster was formed and poured, and five to eight days later the concrete was placed for the 8-in. thick reinforced concrete ceiling slab. While curing, this slab was supported by temporary forms resting on jacks. The finished slab completely covered the masonry cavity wall room and projected 1 ft beyond the front (west) cavity wall. At the time of placing the concrete for the pilaster, the RCMU non-responding walls were filled with cement grout.

The top course of the masonry cavity wall consisted of solid top block and the cavity between the brick and CMU wythes was spanned by a 4-1/2-in. wide strip of sheet metal nailed to the masonry to prevent intrusion of the concrete from the ceiling slab into the cavity. The only bond between the ceiling slab and its supporting walls was that provided by fresh concrete placed on masonry; no mechanical ties were used.

The clear distance between floors was 8 ft 8 in.

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FIGURE 8   CONSTRUCTION OF RCMU NON-RESPONDING SUPPORT WALLS

FIGURE 9   CONSTRUCTION OF MASONRY CAVITY TEST WALL

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Fachwerk (Half-Timber) Framing

After the aboveground concrete work was complete, carpenters began construction of the timber framework for the Fachwerk walls shown in elevation in Figure 2. Although most of the components were prepared in the contractor's shop in Socorro, it was found necessary to fit some on the job after the major portions of the framwork had been put in place. All joinery was mortise and tenon, illustrated in Figure 10. Holes for the pegs were drilled with the timbers joined; tight Douglas fir pegs were then driven in with a plastic mallet. Figure 11 is a photograph of the partially completed timber work on the south wall. The bolts seen in the foreground of Figure 7 are seen again in Figure 11 holding the bottom plate in position (a similar series of bolts fastens the west Fachwerk wall). The projecting portion of the wood pegs were cut off after final adjustments in timber positions were made. The tenon joining a horizontal brace to a diagonal brace is seen in the upper left of the photograph.

As noted above, the Fachwerk test walls bottom plates were anchored to the floor slab. In addition there were four anchor bolts holding vertical timber posts to the non-responding atruccural elements: two bolts in the pilaster at the north end of the west front wall and two bolts in the 12-in. RCMU at the east end of the south side wall. Since the timber framing was fastened together with mortise and tenon joinery, it was impossible to cast all these bolts into the masonry or concrete. Therefore the bolts projecting from the horizontal surface of the floor slab were cast in place while those on vertical surfaces were expansion type bolts driven into drilled holes through the in-place timber posts. During construction it was found possible to use cast-in-place bolts in vertical surfaces if they were sufficiently high above the horizontal surface, hence on Structures Nos. 2 and 3 some of the upper bolts are cast-in-place.

Ceiling and Roof Framing

With the south and west Fachwerk framing in place, the carpenters erected 6-by-10-in. ceiling joists, one of which served also as the top plate in the Fachwerk south wall. The tenons seen at the top of the posts in Figure 11 fit mortises in this joist.

The joists are held to the top plate of the Fachwerk west wall by 3/4-in. deep notches and by oak dowels 1-1/2 in. in diameter by 5 in. in length. Like the concrete overhead slab, the joists extended beyond the west front wall by 1 ft. At their eastern (back) ends the joists were nailed to a wooden plate bolted to the RCMU.

A false ceiling of 5/8-in. thick plywood was nailed between the joists to carry a 3-1/2-in. deep layer of brick, pictured in Figure 12. The ceiling brick were painted white on the top surface to distinguish them in any post-shot debris from structural brick. After the brick were painted, the attic floor was surfaced with 3/4-in. thick plywood.
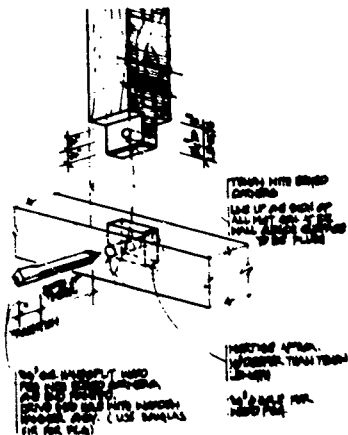
FIGURE 10   TYPICAL MORTISE AND TENON JOINT USED IN FACHWERK FRAMING
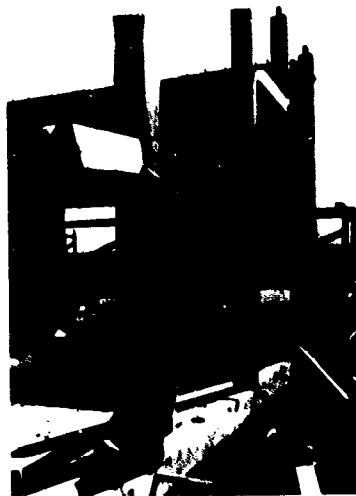
FIGURE 11   PARTIALLY COMPLETED TIMBER FRAMING
FOR FACHWERK SOUTH SIDE WALL

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FIGURE 12   BRICK IN PLACE BETWEEN CEILING JOISTS

Roof erection began with the two 4-by-6-in. rafters at the north end of the structure and the 6-by-6-in. ridge beam supported on 6-by-6-in. posts. The ridge beams over the two rooms were separate. Rafter spacing was approximately 24 in. on center over the north (cavity wall) room and 30 in. on center over the south (Fachwerk) room and sloped at an angle of 51 degrees to the horizontal. Rafters were fastened at the ridge and at the bottom plate with 8-in. spikes driven into pre-drilled holes. As soon as the roof framing was well underway, the masons were able to continue the north cavity wall in the gable on top of the ceiling slab. In Figure 13 the masons have just mortared the first two CMU courses on top of the reinforced concrete ceiling slab. The attachment of rafter to plate and the bolting of plate to slab is also shown in the photograph; the attachment of rafter to ridge beam was similar to its attachment at the plate. Ridge beam supporting posts were toe nailed at the top into the ridge beam and at the bottom into special plates bolted to the slab. Figure 14 shows an overall view of the work at this stage.

Fachwerk Brick Masonry

While the carpenters were finishing the roof framing and the Fachwerk south gable framing, the masons began filling the half-timber framework. One wythe of common brick in running bond was mortared into each subpanel. In addition to the mortar placed between brick and timber members, 50d nails half driven into the wood and half embedded in approximately every fifth horizontal mortar joint also provided bonding between masonry and timber. Figure 15 is a view from inside the structure showing the masons at work on the first story Fachwerk. The 50d nail at the first course can be seen in the lower central portion of the scene. The plywood false ceiling has been installed between joists. Note that the brick are laid flush with the timber framework on their inner surfaces. Because of the difference in thickness between the brick and timber the outside surfaces are not flush but the brick is substantially inset. In the Fachwerk gable however nominally 4-by-6-in. timber was used for framing so that the brick was flush on both surfaces. A view from outside, Figure 16, shows the inset brick in the first story and the flush brick in the gable.

Figure 16 also shows the different rafter spacing over the two rooms. The ridge beam and plates are not continuous but are cut between the two closely spaced rafters (seventh and eighth from the south or right hand side). The three ridge posts are visible supporting the ridge over the Fachwerk room. Three post were planned for the cavity wall room as well but during construction it was noted that the center post would rest on the position of the upward facing pressure gage in the center of the overhead slab. Therefore, two equally spaced posts were substituted for the single central post. In Figure 16 only two of these four posts over the cavity wall room are in place. Note that the northermost post over the Fachwerk room is within a few inches of the southernmost post over cavity wall room.

In Figure 16 can also be seen the pre-cast reinforced concrete header spanning the single masonry cavity wall window. A similar header was

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FIGURE 13   OVERALL VIEW OF CONSTRUCTION OF
MASONRY CAVITY WALL IN GABLE

FIGURE 14   CONSTRUCTION OF MASONRY CAVITY WALL
IN GABLE ABOVE CONCRETE CEILING SLAB

FIGURE 15   INTERIOR VIEW OF FACHWERK MASONRY CONSTRUCTION
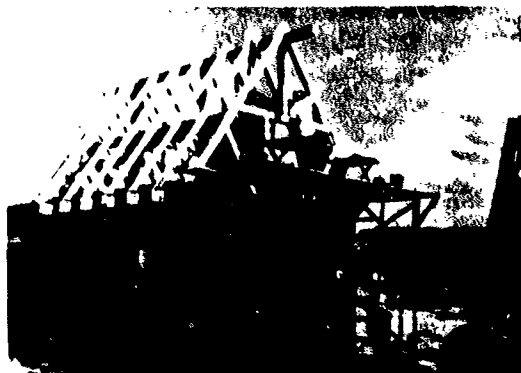
FIGURE 16   MASONRY CONSTRUCTION IN FACHWERK GABLE WALL

built into the CMU wythe of the cavity wall. The sill of this window consisted of two rowlock courses of brick. Wood jambs were nailed to a wood nailing strip tightly wedged into the cavity in the window opening.

All first story windows were glazed with wood window stop holding a single pane of double strength glass; the gable window was not glazed.

Roof Covering

The baked clay roof tile chosen was a European style made in the United States[*] weighing 9.35 psf when laid. Each tile was fastened with a single nail to 1-by-2-in. net wood strips laid parallel to the west wall approximately 14 in. on center. The tile overlapped on all four sides (except for the bottom course and end tier). Figure 17 shows a photograph of the roofing operation. The alternation in tile color was done for photographic reasons. (The Fachwerk panel above the front window was later filled with brick.)

The special closure tile manufactured for use at the ridge was omitted for the test structures; a cap of mortar sealed the ridge instead. The tile nailing strips extended past both north and south gable walls to create roof overhangs of approximately 6 in. on the sides.

TEST RESULTS

Free-Field Overpressure

The air blast data obtained in the vicinity of the three SRI test structures included the free-field measurements along blast line No. 3 at ground ranges of 1140 ft (Structure No. 1) and 2750 ft (Structure No. 3), and measurements in the truck area at 1140 ft, 1370 ft, and 2000 ft. In addition, measurements from the pressure gages on the ground surface adjacent to the north wall of each structure, and from the pressure gages mounted on the front surface of each concrete pilaster could be used to estimate the actual free-field overpressure at each structure location.

For the three structure locations, the discrepancies in the peak free-field overpressure data from four sources are given in Table 1. The values shown in the table as estimated overpressures were obtained by curve fitting the actual data over the ranges of most interest; this was necessary where free-field pressure gages are not located at the same ground range as the structure. The values shown as calculated in the table were obtained by using the initial discontinuity of the reflected pulse measured by the head-on pressure gages mounted on the concrete pilaster of each structure. Although the actual free-field overpressure level at each structure cannot be determined with certainty, it is apparent that the overpressure level was slightly less than the predicted value of 7.0 psi for Structure No. 1, was approximately equal to the 3.5 psi predicted value for Structure No. 2, and was a little below the 2.0 psi predicted value for Structure No. 3.

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[*] Ludowici-Celadon Company, Chicago, Illinois.

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FIGURE 17   TILE ROOFING OPERATION

Table 1

PREDICTED AND MEASURED PEAK FREE-FIELD OVERPRESSURES
AT LOCATIONS OF THREE SRI TEST STRUCTURES

Range (ft)	Peak Overpressure (psi)				
	Predicted	Blast Line #3[†]	Calculated[*]	Outside North Wall	Truck[†] Area
1140	7.0	6.3	6.8	6.7	6.7
1730	3.5	3.8[**]	3.3	3.7	
2750	2.0	2.5	1.7	1.8	

[*] Calculated from peak reflected peak overpressure at pilaster gage (Γ-2), assuming perfect reflection. Assumed ambient air pressure = 12.325 psi.

[†] Values reported by letter dated 21 April 1977 from Test Group Director, Dice Throw.

[**] Estimated.

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Post-Shot Damage Survey

The post-shot damage survey started immediately after the test bed was cleared for re-entry. The overall damage to the three test structures can be briefly summarized as follows. Structure No. 1, as shown in Figure 18, experienced a catastrophic collapse of the front cavity wall and a portion of the side cavity wall, and of the front and most of the side Fachwerk wall panels. Structure No. 2, as shown in Figure 19, experienced structural damage to the front cavity wall, although the wall was still in place, and cracking of the side cavity wall. The Fachwerk walls had a partial collapse of the 4-ft wide masonry panels on both the front and side walls, as well as damage to some of the timber members. Structure No. 3, as shown in Figure 20, had only hairline cracking in the cavity walls, and some cracking in the Fachwerk masonry panels.

Figure 18 - 20 also show the post-shot damage to the roof systems of the three test structures. It was apparent from the test that the roof tile was stronger in bending than originally estimated, and the roof framing therefore received much higher dynamic loading than anticipated. This resulted in much greater damage to the roof support members: at Structure No. 1 all front rafters, except the one adjacent to the north cavity wall gable, were fractured; at Structure No. 2 all front rafters, except the two end rafters, were fractured or experienced horizontal splitting; and at Structure No. 3 three front rafters had horizontal splits and three had lifted from the front plate. Details of the damage to each structure is given in the following sections.

Structure No. 1

Masonry Cavity Wall. As can be seen in Figure 21 the brick and CMU on the west front wall were completely collapsed. On the front wall, the only remaining CMU was the bottom course and a portion of the top course that was still adhered to the concrete ceiling slab (Figure 22). The remaining brick on the front wall was a 27-in. vertical section, adjacent to the concrete pilaster, that had rotated inward about 1 ft, three courses of brick along the front above the floor slab, and the top course of brick still bonded to the concrete ceiling slab.

The north side cavity wall was collapsed for a distance of about one third its length from the front wall as shown in Figure 23. The bottom two courses of CMU, at their intersection with the front wall, were still in place although they had rotated outward and forward about 3/4 in. (Figure 24). Portions of the brick and CMU that were still standing had the Z-ties intact even though the two mythes had rotated about 4 in. outward as a unit (Figure 25). The top course of brick and CMU in the side wall were still bonded to the concrete ceiling slab in the same manner as described for the front wall.

The fracture planes in the top course of CMU and brick in the front wall below the ceiling slab joint are shown in Figure 26; the fractures

25
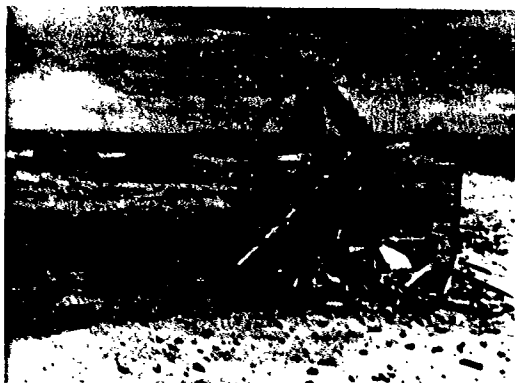
FIGURE 18a POST-SHOT VIEW OF STRUCTURE NO. 1
FROM SOUTH FACHWERK SIDE

FIGURE 18b POST-SHOT VIEW OF STRUCTURE NO. 1 FROM
NORTHWEST MASONRY CAVITY WALL SIDE

FIGURE 19   POST-SHOT VIEW OF STRUCTURE NO. 2
FROM SOUTHWEST FACHWERK SIDE

FIGURE 20   POST-SHOT VIEW OF STRUCTURE NO. 3
FROM SOUTHWEST FACHWERK SIDE

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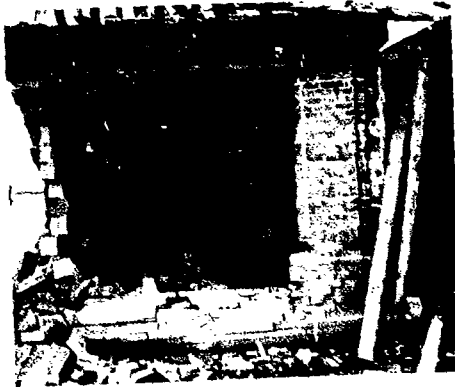
FIGURE 21   COLLAPSED FRONT MASONRY CAVITY WALL STRUCTURE NO. 1
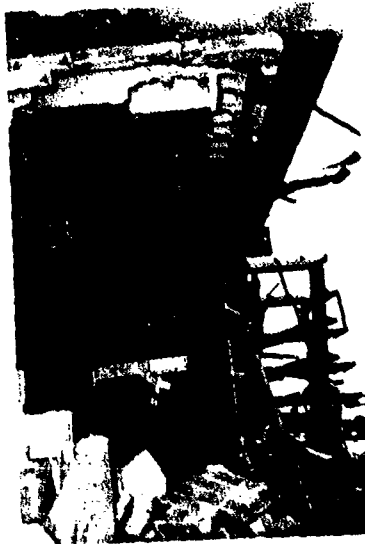
FIGURE 22   END VIEW OF COLLAPSED MASONRY
CAVITY WALL STRUCTURE NO. 1

FIGURE 23   PARTIALLY COLLAPSED NORTH SIDE
MASONRY CAVITY WALL STRUCTURE NO. 1

FIGURE 24   FRACTURE AT INTERSECTION OF WEST FRONT AND NORTH
SIDE MASONRY CAVITY WALLS STRUCTURE NO. 1
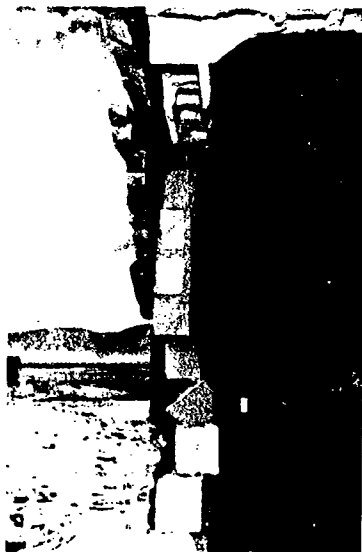
FIGURE 25   OUTWARD ROTATION OF NORTH
             MASONRY CAVITY WALL STRUCTURE NO. 1

FIGURE 26   FRACTURE PLANES IN TOP COURSE OF CMU IN FRONT CAVITY
             WALL BELOW CONCRETE CEILING SLAB STRUCTURE NO. 1

were similar in the side wall. The fracture pattern on the inner face of the CMU indicates a shear/compression failure, whereas that on the outer face of the CMU indicates a tensile failure. The failure at the mortar joint between bricks, which can also be noted in Figure 26, indicates that the brick wythe failed in tensile bond.

At the connection between the front cavity wall and the concrete pilaster, the fracture pattern in the CMU was as just described for the top course of the CMU at the ceiling slab. The portion of the brick wythe still standing in the front wall exhibited a typical tensile fracture along a vertical section one-half brick in from the concrete pilaster support (Figure 21). A damaged section of the brick, on an approximate diagonal from the upper right corner, indicated compressive spalling.

The bottom course of the CMU in the front wall was still in place in front of the kicker plate. However, as shown in Figure 27, the inner face of the CMU was fractured vertically through the webs, and the top edge of the inner face had rotated inward about 1 in. The remainder of each block was still in place, although the bond between CMU and the bottom mortar bed was broken. Removal of the inner face indicated that the bottom course of CMU in the front wall had also failed in shear/compression at the inner face. The brick along the bottom of the collapsed front wall had failed in tension at the horizontal mortar-to-brick joint.

An examination of the steel Z-ties placed between the brick and CMU wythes showed that many of the ties were undamaged. Of the ties found in the debris, only a few were bent, one was fractured, and one apparently experienced an axial column type buckle. The remainder of the ties found in the debris, as well as those remaining in the damaged walls, appeared in their original condition. It's therefore possible that some of the ties found in the debris were damaged during collapse of the wall.

Because of the condition of the Z-ties, e.g., Figure 28, and because the brick and CMU in damaged but still standing sections of the walls had rotated as a unit (e.g., Figure 25), it is apparent that the steel Z-ties placed on 16-in. centers horizontally and vertically had sufficient strength to transfer the blast forces between the outer brick and inner CMU wythes. As a result of the two wythes acting as a unit, rather than as individual panels, the masonry cavity wall developed a greater resistance to the blast forces than assumed in the analysis. A cursory examination of the test data indicates that the measured velocity of the cavity wall at time of collapse was about two-thirds of the predicted velocity. On the other hand, from the measured wall velocity at collapse, as well as from the distribution of the front wall debris, it is apparent that the free-field overpressure of about 6.5 psi at Structure No. 1 was, as predicted, well above the incipient collapse overpressure for the front masonry cavity wall.

Fachwerk Wall. As can be seen on Figure 29 the brick and timber panels of the west front Fachwerk wall were completely collapsed. The only members remaining near their original positions were the vertical post adjacent

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FIGURE 27   FRACTURE IN BOTTOM COURSE OF CMU IN
             FRONT CAVITY WALL STRUCTURE NO. 1

FIGURE 28   TYPICAL INTACT Z-TIE EMBEDDED IN BRICK MASONRY
             FROM FRONT MASONRY CAVITY WALL STRUCTURE NO. 1

FIGURE 29   COLLAPSED WEST FRONT FACHWERK WALL STRUCTURE NO. 1

to the concrete pilaster and the bottom plate which was fastened with anchor bolts to the concrete floor slab.

On the south side wall, only two of the four Fachwerk panels remained standing; as shown on Figure 30 the two remaining panels were badly damaged. With the collapse of most of the vertical timber supports in the Fachwerk walls, the ceiling and roof systems collapsed onto the floor slab.

The collapse of the Fachwerk wall panels involved at least three primay mechanisms; (1) the deflection and break-up of the brick masonry, (2) the large deflection and fracture of the timber posts, and (3) fracture of the plates at the mortise and tenon connections. There were also other identified collapse mechanisms, which were considered as minor or secondary. An example of a minor mechanism would be the pullout or fracture of the wooden pegs in the mortise and tenon joints. An example of a secondary collapse would be the large vertical deflection of the top plate of the front wall, after the brick and timber posts had collapsed, and the subsequent fracture of the corner post between the front and side Fachwerk walls. An examination of the damage on both Structures No. 1 and 2 (to be discussed subsequently), as well as a preliminary review of the high-speed test movies, indicates that the three mechanisms occurred simultaneously, or if they occurred in a definite sequence it was not apparent from the information examined to date. It is thought at present that if there was a preference for a specific collapse mechanism to occur, then it was more a function of flaws in the materials or construction than large differences in basic structural strength.

The final disposition of the debris in the interior of the room from the front Fachwerk wall panels, such as shown in Figure 31, indicates that the 6.5 psi overpressure level was well above the incipient collapse overpressure of the wall. The timber elements of the front wall were distributed towards the rear of the room, and in several cases were found deposited against the rear RCMU wall. For example, Figure 32 shows the timber post from the left (north) of the window resting against the rear wall with the lintel still attached. The bottom of the post is upward indicating that the post rotated during translation, and timber marks on the wall indicate that the post impacted the rear wall with considerable velocity. Also, the major portion of the bricks from the front wall were found close to the rear wall; in fact, one brick impacted the rear RCMU wall 41 in. above the floor slab, as shown in Figure 33.

A preliminary review of the test movie for Structure No. 1 indicated that as the inward center deflection of the timber posts of the front wall became large, the mortise joints in the bottom plate fractured, except for those at the two end posts. With removal of the bottom support for the posts, the wall panels rotated inward at the bottom under the blast forces. When this occurred, the top plate of the front wall was essentially spanning the distance between the two end posts. Under the influence of the high roof dead loads, and possibly some blast forces, the top plate deflected downward at the center placing an eccentric load on the corner post between the west front and south side Fachwerk walls. The corner post fractured at the top and collapsed outward from the corner.

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FIGURE 30   TWO PANELS REMAINED STANDING IN SOUTH
             SIDE FACHWERK WALL STRUCTURE NO. 1

FIGURE 31   INTERIOR DEBRIS FACHWERK TEST CELL STRUCTURE NO. 1
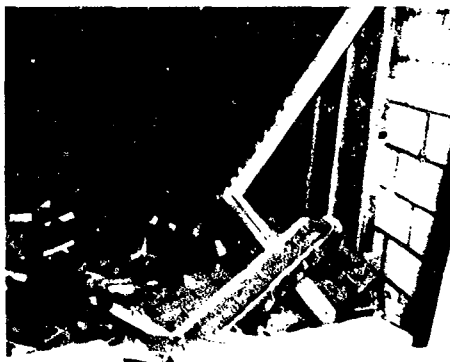
FIGURE 32 INTERIOR DEBRIS FACHWERK TEST CELL STRUCTURE NO. 1

FIGURE 33 BRICK FRAGMENT EMBEDDED IN REAR RCMU WALL 41 INCHES
ABOVE FLOOR SLAB AS A RESULT OF IMPACT OF BRICK FROM
COLLAPSED FRONT FACHWERK WALL STRUCTURE NO. 1

The initial collapse of the south side Fachwerk wall was similar to the front wall, however, the damage was not nearly as great and the final disposition of the members was different. The test movie indicated that as the inward deflection at the center of the vertical post to the left of the 48-in. wide panel (the first vertical post east of the front wall corner post) became large, the mortise joint in the bottom plate fractured, and the wall panel started to rotate inward at the bottom. However, because the net inward blast force on the side wall was much less than that on the front wall, and even reversed direction during the load cycle, the side wall did not collapse inward as the front wall had done, but rather the inward deflection reached a maximum and then decreased. With the collapse of the corner post, as described above, the two damaged panels in the side wall (those nearest to the front wall) collapsed, with most of the debris being deposited on the outside of the structure.

Roof System. As shown in Figure 29 the roof system over the Fachwerk wall test cell was completely collapsed, and that over the cavity wall test cell was severely damaged but still in place. All roof tile had been removed by the blast and was displaced as far as 121 ft behind Structure No. 1, and 71 ft in front of the structure.

The roof framing over the Fachwerk wall had collapsed onto the test cell as a result of the wall collapse. However, even though all front roof rafters had been fractured by the blast loading, all rear rafters were structurally intact and still nailed to the rear plate (Figure 30).

The roof framing over the cavity wall test cell was approximately in position although skewed, as noted in Figure 34. All front rafters, except the one adjacent to the north side cavity wall gable, had experienced typical flexural fractures near the center of the rafter span. The top portions of the rafters were still nailed to the ridge beam, but were hanging downward, and the bottom portions had rotated around the front plate and were resting on top of the reinforced concrete ceiling slab. Figure 34 also shows that the six rear rafters, four ridge beam posts, and the ridge beam above the masonry cavity wall are all structurally intact but somewhat displaced.

Structure No. 2

Masonry Cavity Wall. Although the front cavity wall was still in place, there was extensive cracking of the exterior wythe of brick as shown in Figure 35. As noted for Structure No. 1, there was also a vertical crack one-half brick in from the concrete pilaster support in Structure No. 2. Another vertical crack was apparent at the juncture of the front and side cavity walls; this crack extended the full height of the wall and was continuous in either the front or side walls. The crack width varied from a small crack at the bottom to about 1/4 in. near the top (Figure 36). The diagonal cracks from the left corners of the wall shown in Figure 35, showed some compressive spalling. It appears that the window jambs wedged

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FIGURE 34 DAMAGED ROOF FRAMING ABOVE MASONRY
          CAVITY WALL TEST CELL STRUCTURE NO. 1
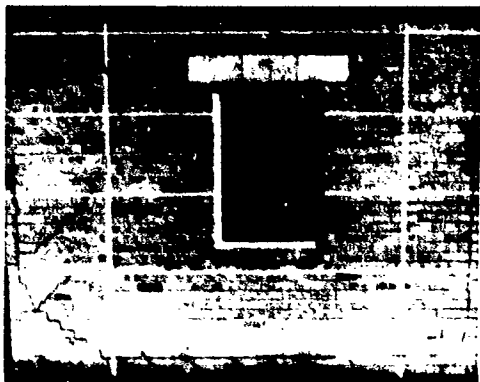
FIGURE 35   CRACKED FRONT MASONRY CAVITY WALL STRUCTURE NO. 2
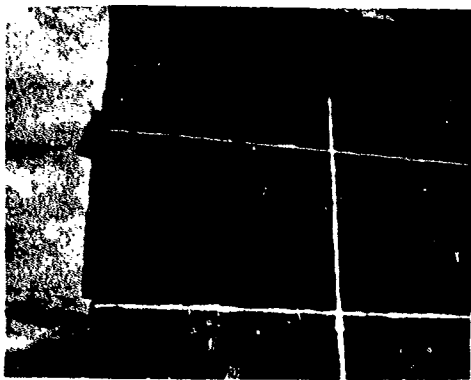
FIGURE 36   CRACK AT CORNER OF FRONT AND SIDE
            MASONRY CAVITY WALLS STRUCTURE NO. 2

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in the cavity between the two wall wythes provided sufficient stiffness to the wall at the window opening to modify an otherwise fairly classical crack pattern.

Figure 37 shows the crack pattern in the exterior of the north side cavity wall of Structure No. 2; the juncture of the side and front walls is on the right in the figure. Except for the crack at the corner, the cracks in the wall vary in width from about 1/16 in. on the right part of the wall to hairline on the left. Although not discernible in the figure, a portion of the wall above and to the right of the center in Figure 37 was pushed inward about 1/4 in. from its original position.

An examination of the interior surface of the CMU indicated that there was more extensive damage to the front cavity wall than was apparent from the cracks in the exterior surface of the brick. The inner face of the top course of CMU above the window had fractured just below the concrete ceiling slab for a horizontal distance of about 5 ft. The CMU below the fracture had a maximum inward permanent displacement of about 1/2 in. near the center of the fracture as can be seen in Figure 38; the displacement decreased to zero near the ends of the fracture. Also, the portion of the block above the fracture plane was still well bonded to the bottom surface of the concrete ceiling slab. The fracture plane on the inner face of the CMU in the front wall of Structure No. 2, which is shown in Figure 38, was identical to that observed in a like location in Structure No. 1 (see Figure 26).

In addition to the fracture above the window, there was also a gap at the intersection of the CMU in the front and side walls. Figure 39 shows that the side wall, on the right in the photograph, has separated and displaced about 1/2 in. away from the front wall; the figure shows the gap in the second course of CMU below the concrete ceiling slab.

In order to examine for further possible damage to the CMU in the front wall, the window frame was removed to permit inspection of the cavity between the brick and CMU wythes. It was discovered that the horizontal mortar joint under the top course of CMU had failed and that the CMU wythe below the top course had displaced inward about 1/2 in. relative to the top course of CMU, which was obviously still bonded to the concrete ceiling slab. It was therefore apparent that the inner face of the front wall CMU in Structure No. 2 had experienced a compression/shear type of failure and that the top block had split along the webs and horizontal mortar joint in a manner similar to that noted for the bottom course of CMU in the front cavity wall of Structure No. 1 (see Figure 27).

Because of the damage to the front cavity wall CMU just described, the rear surface of the brick wythe was examined in the cavity to determine why the exterior surface of the brick had given no indication of the degree of damage sustained by the CMU above the window. A close examination of the rear brick surface showed that although some additional cracks in the brick were found, there was no evidence that the brick wythe had a permanet displacement of 1/2 in. The top course of bricks was well bonded to the concrete ceiling slab, and there was no detectible slippage along any of the

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FIGURE 37   CRACKED SIDE MASONRY CAVITY WALL STRUCTURE NO. 2

FIGURE 38 FRACTURE IN INNER SURFACE OF CMU IN FRONT CAVITY WALL
JUST BELOW THE CONCRETE CEILING SLAB STRUCTURE NO. 2
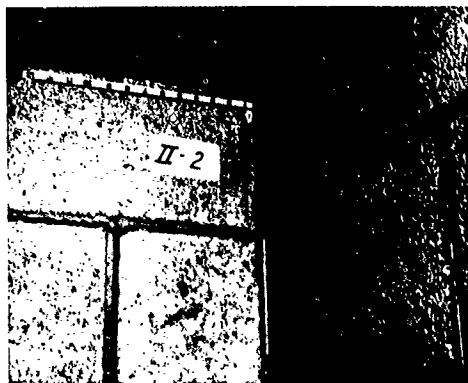
FIGURE 39 GAP AT INTERSECTION OF CMU IN FRONT AND SIDE
MASONRY CAVITY WALLS STRUCTURE NO. 2

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mortar joints. However, it was found that some of the Z-ties between the two wythes had slipped in the mortar joints, permitting the two wythes to separate.

From the similarities between the damage sustained by the front cavity wall in Structure No. 2 and the failure modes observed in identical locations in Structure No. 1, it is apparent that the free-field overpressure of about 3.5 psi at the Structure No. 2 location was sufficient to develop the maximum resisting capability of the cavity wall. Furthermore, it is felt that the wall was close to its collapse overpressure, and that an increase in overpressure of as little as 1/2 psi would have produced significantly greater wall displacement and a correspondingly greater damage.

The interior CMU of the north side cavity wall showed crack patterns similar to, but more extensive than, those observed in the exterior brick. Also, the cracks in the CMU were generally wider than found in the brick, being as wide as 1/8 in. in the area towards the front of the wall.

Fachwerk Wall. A view of the damage to the west front Fachwerk wall of Structure No. 2 can be seen in Figure 40. As can be seen in the figure, the masonry in the 48-in. wide panel to the left of the window has been severely damaged with a portion having collapsed onto the floor slab, the masonry in the panel below the window has collapsed, and there is damage to the masonry in some of the other panels. In addition, the tenons are exposed at the bottom of the post and diagonal to the right of the window, indicating a failure of the bottom plate at the mortise joint, and there are splits in the top plate at the joints for both the post and diagonal to the right of the window.

The damage to the south side Fachwerk wall is shown in Figure 40. Although the damage to the 48-in. wide panel to the left of the window appears somewhat similar to that observed for the identical panel in the front wall, the damage to the other masonry panels and timber framing is much less than that observed on the front wall at the same location.

Figures 41 and 42 show in more detail the damage sustained by the west front Fachwerk wall. It is apparent that the bottom plate at the mortise joint to the right of the window fractured, permitting the post and diagonal to displace inward at the bottom with a permanent deflection of about one ft; this deflection probably accounts for the loss of brick below the window. The fracture in the plate is shown graphically in Figure 43, and is a typical failure mode, noted in both Structure Nos. 1 and 2.

The post to the left of the window in Figure 42 split parallel to the grain for its entire height; this is also a typical failure mode often observed in Structure No. 1. The post to the left of the 48-in. wide brick panel, although still in place, sustained a classical bending failure, exhibiting compressive distress on the exterior surface and tensile fracture on the inner surface. In addition, cracking of the inner surface of the bottom plate in the vicinity of the mortise joint for this post indicated

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FIGURE 40   DAMAGED WEST FRONT AND SOUTH SIDE
FACHWERK WALLS STRUCTURE NO. 2

FIGURE 41   DAMAGED WEST FRONT FACHWERK WALL STRUCTURE NO. 2

FIGURE 42 OBLIQUE VIEW OF DAMAGED WEST FRONT FACHWERK WALL STRUCTURE NO. 2

FIGURE 43 TYPICAL FRACTURE IN BOTTOM PLATE AT MORTISE JOINT WEST FACHWERK WALL STRUCTURE NO. 2

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that a fracture of the plate had been initiated which was similar to that observed at other bottom plate mortise joints.

The brick masonry in the 48-in. wide panel on the front Fachwerk wall, shown in Figure 44, was approximately one-half collapsed with the remainder severely damaged. The masonry showed evidence of failure at the mortar joints and compressive spalling of the exterior surface of the bricks.

The final disposition of the debris from the front wall is shown in Figure 45. Although some debris was translated as far as the rear RCMU wall, none was piled against the rear wall and no brick impacted the rear wall above the floor slab as occurred in Structure No. 1. This indicates, of course, a lower brick velocity at time of collapse for Structure No. 2 than for Structure No. 1.

The damage to the south side Fachwerk wall is shown in Figure 46. As can be seen in the figure, the damage to the 48-in. wide brick panel on the side wall is different from that on the front wall in that a large section of the side wall panel, below the collapsed mid-section, is intact although rotated inward at the top. This type of failure mode was probably a result of failing to install one of the masonry-to-timber nails in the right hand post at about the mid-height of the rotated section. The masonry above and below the collapsed portion sustained considerable cracking and spalling of the brick similar to that noted in the front wall panel.

Of the 48 bricks comprising the eight collapsed courses of brick in the 48-in. wide panel in the south side wall, half of the bricks fell on the interior floor slab of the structure and half on the exterior.

The only damage to the side wall timber framing was a vertical split in the post support to the left of the 48-in. wide panel. Although the post was still in place, the split extended from the outer face of the post just below the horizontal brace diagonally upward to the rear face of the tenon at the top of the post.

Roof System. As shown in Figure 40, the front roof system of Structure No. 2 sustained considerable damage. Of the 13 front rafters only the two end rafters adjacent to the north and south gable walls were not damaged. Above the cavity wall test cell five of the front rafters experienced typical flexural fractures near the center of their spans. Of the seven front rafters above the Fachwerk wall test cell, three had flexural fractures, and three had horizontal splitting, but did not fracture. Figure 47 shows an interior view of the damage to the front roof system for Structure No. 2. None of the rafters in the rear portion of the roof were damaged.

Figures 40 and 48 show the damage to the roof tile. Each half of the roof had 12 rows of 43 tiles for a total of 516 tiles. As shown in Figure 40, the front tile was damaged in all sections of the roof, but damage was

FIGURE 44   DAMAGED 48-IN. WIDE PANEL ON FRONT
            FACHWERK WALL STRUCTURE NO. 2
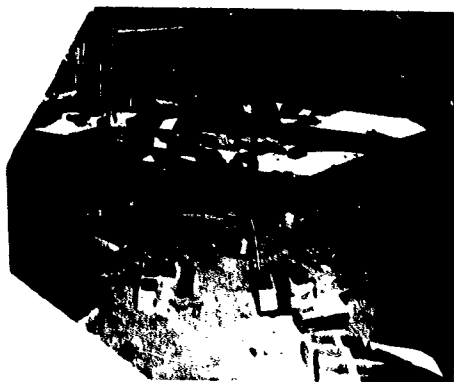
FIGURE 45   DISTRIBUTION OF BRICK DEBRIS FROM FRONT
            FACHWERK WALL INTERIOR STRUCTURE NO. 2

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FIGURE 46   DAMAGE TO SOUTH SIDE FACHWERK WALL STRUCTURE NO. 2
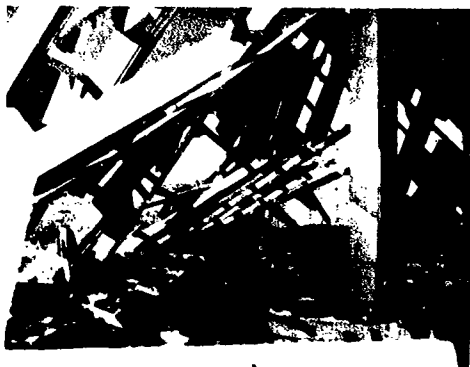
FIGURE 47   INTERIOR VIEW CF FRACTURED FRONT
            ROOF RAFTERS STRUCTURE NO. 2
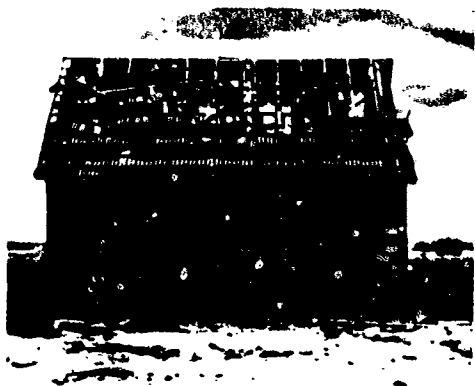
FIGURE 48   REAR VIEW OF DAMAGE TO ROOF TILE STRUCTURE NO. 2

concentrated in the lower courses. A total of 73 percent of the tiles were broken or missing from the front surface. As shown in Figure 48, on the rear half of the roof, in contrast to the front, there was much more damage to the tile on the upper portion of the roof than towards the bottom; in fact, the top three courses down from the ridge were completely destroyed. Although the overall tile damage appears less on the rear than on the front roof, 68 percent of the rear roof tiles were broken or missing.

Structure No. 3

Masonry Cavity Wall. There was only minor damage to the masonry cavity walls of Structure No. 3, shown in Figure 49. The only noticable crack was a 1/32-in. wide horizontal crack in the brick mortar joint one brick down from the top of the west front cavity wall. All other cracks in the brick and CMU wythes were small or hairline.

Fachwerk Wall. Figure 49 shows the west front and south side Fachwerk walls. Although there was no noticeable damage to any of the Fachwerk timber framing in these walls, there was some cracking of the masonry. In the 48-in. wide panel on the west Fachwerk wall, there was a large crack in the brick masonry to the left of the window opening. As shown in Figure 50, the portion of the wall to the right of the diagonal cracking is displaced inward about 1/4 in. relative to the left portion. Also, the center portion of the masonry panel had about a 1/2-in. permanent inward deflection. Some of the other front wall masonry panels had minor cracks, but all were less than 1/64-in. wide.

The south side Fachwerk wall in Structure No. 3 was generally in good condition with only small or hairline cracks in some of the brick masonry.

Roof System. As shown in Figures 49 and 51 the roof system received only minor damage in Structure No. 3. Of the 13 rafters on the front of the roof, one of the rafters above the cavity test cell and three above the Fachwerk test cell experienced horizontal shear failures near midspan. In addition, three rafters near the center of the structure lifted above the front roof plate about 3/4 in. There was no noticable damage to any of the other roof framing members.

Figures 49 and 51 show the minor damage to the roof tile. Of the 516 tiles on the front roof surface, 20 percent were broken or missing As noted previously for Structure No. 2, the tile damage for Structure No. 3 was also concentrated in the lower courses on the front roof and the upper courses on the rear roof.
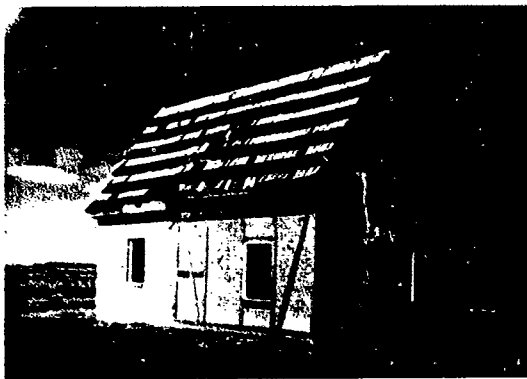
FIGURE 49   POST-SHOT VIEW OF STRUCTURE NO. 3
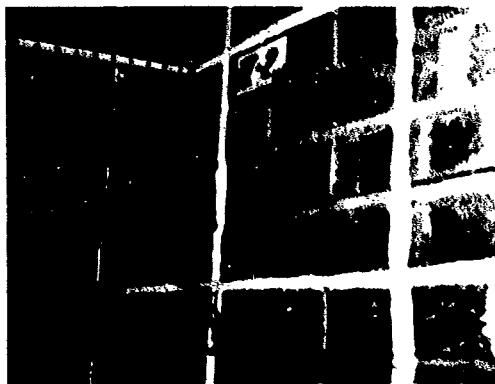
FIGURE 50   DAMAGED 48-IN. WIDE PANEL ON FRONT
FACHWERK WALL STRUCTURE NO. 3
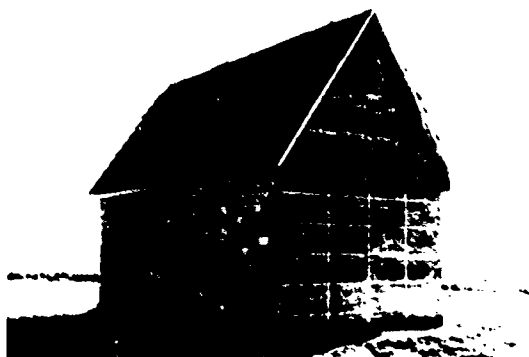
FIGURE 2. DAMAGE T ... ... TILE STRUCTURE NO. 3

Electronic Gage Data

For the purpose of measuring (a) air blast loading on test walls and (b) movement or response of these walls, each of the three structures was instrumented with eight pressure and three displacement gages. Two of the pressure gages were outside and the remainder inside the building. All displacement gages were attached to the inner surfaces of walls, one to each of the front and side masonry cavity test walls and one to a single panel of the front (west) Fachwerk wall (4-ft wide panel adjacent to window). Of the two outside pressure gages mounted at each structure one was a total head gage installed in the reinforced concrete pilaster in the front (west) wall facing ground zero; the second gage measured side-on overpressure at the ground surface just outside the north wall. The structure most remote from ground zero (Structure No. 3) contained two additional total head pressure gages in the concrete pilaster. The inside pressure gages measured side-on pressures in floors and ceilings.

All gage information was transferred by cable as FM signals to recording equipment in Instrumentation Park No. 3 approximately 6000 ft east of ground zero. Data reduction was begun by playback through a discriminator to a visicorder, the record from which was digitized on a Benson-Lehner 29E Telereadex by an operator who aligned crosshairs on a magnified image of the visicorder trace. The same operator also read calibration steps which had been placed on a separate magnetic tape approximately one week before the detonation.

Simultaneously with the pressure and displacement data, timing information (in IRIG B format) was directly recorded on the magnetic data tape. From this signal absolute times of first response at each gage were read to half a ms. Times relative to the detonation were found from the reported time of detonation: 8 hours 0 minutes 0.048 seconds local time. A summary of the performance of and peak values recorded by the electronic gages is contained in Table 2. Most of the records are noisy, but only two failed to produce useable results.

Cameras

Although the difficult lighting conditions reduced the resolution of the pictures at all structures, only one camera failed to produce useable footage. The northwest camera at Structure No. 1 did not start. All other cameras provided acceptable coverage from the moment of detonation until the negative wind phase brought dust from immediately behind the structure forward to obscure the camera view. By that time the structural response was well defined. Because of the relatively short positive wind duration at the 7 psi free-field overpressure level, this obscuration allowed a shorter viewing time at Structure No. 1 than at the other two structures.

The approximately one millisecond interval between frames permits good temporal resolution of events. Clearly visible are shock arrival and passage over the structure, window breaking, inward deflection of both front and side Fachwerk walls and of front masonry cavity walls, downward

Table 2

ELEKTRONIC GAGE RESULTS

Kind of Gage	Location	Structure No. 1		Structure No. 2		Structure No. 3	
		Peak	Comment	Peak	Comment	Peak	Comment
PS	1/4 floor 5k corner	6.2 psi	2-step rise	3.2 psi	Mild ringing; 3-step rise	1.7 psi	
PS	1/4 floor under window	5.0 psi	2-step action	2.8 psi	Ringing; vortex action	1.5 psi	
D	1/4 west wall	4.0 in.	Inward ends @ 22 ms with saturation	6.0 in.	Full range reached at 32 ms	3.7 in.	Never outward but oscillates about permanent deflection ~ 0.5 in.
PS	C/4 ceiling over window	4.3 psi	Strong vortex action; ringing	2.9 psi	Ringing; vortex thin	1.3 psi	
PS	1/4 ceiling center down	2.5 psi	Disturbed rise; strong ringing	1.0 psi	Ringing	1.1 psi	
PS	ceiling center up	7.2 psi	3-step rise; random noise	3.3 psi	Ringing	2.4 psi	
PS	C/4 floor inside 5 wall	4.0 psi	Multiline structure on rise	2.5 psi		1.3 psi	
PS	exterm outside 5 wall	6.2 psi	Secondary peak (overtest?)	3.7 psi	Secondary peak	1.8 psi	Secondary peak
D	C/4 west wall	6.0 in.	Saturates @ 68 ms but reverses; goes negative	6.8 in.	Permanent deflection 0.1 in.; never outward	.09 in.	Permanent deflection 0.03 in.; never outward
A	C/4 north wall	+1.6 in. −2.6 in.	Permanent deflection −0.5 in.; strong oscillation	+0.4 in. −0.2 in.	One-cycle oscillation; no permanent deflection	.07 in.	Oscillation
PS	Pilaster middle	16.7 psi		7.4 psi		3.6 psi	Record ends @ 70 ms
PS	Pilaster bottom	---		---		3.4 psi	Record ends @ 65 ms
PS	Pilaster top	---		---		3.3 psi	

PS = Sidewall overpressure
P = Inlet overpressure
D = Deflection
1/4 = Internal room
C/4 = Masonry cavity wall room

deflection of the front top plate at Structure No. 1, and stripping of roof tile. At Structure No. 1, the outward collapse of the Fachwerk side wall is just beginning when dust obscuration begins; the inward collapse of the front masonry cavity wall is also not completely visible. Unfortunately, the behavior of the north side cavity wall at Structure No. 1 is not seen at all because of the camera failure.

Deflection Gages

Every deflection gage provided useable information. Those at the front walls of Structure No. 1 quickly reached full range of 6 in. and were destroyed by the wall debris. The gage in the north masonry cavity wall of Structure No. 1 shows at least one cycle of unmistakeable oscillation, first inward approximately 1-5/8 in., then outward approximately 2-1/4 in. with a period of approximately 300 ms; there is a permanent outward deflection of approximately 1/2 in. The portion of the north wall to which the deflection gage was attached at Structure No. 1 was still standing after the explosion (e.g., see Figure 25).

At Structure No. 2, maximum masonry cavity wall deflections were approximately 3/4 in. and 1/2 in. with a single oscillation and no permanent deflection in the north wall. The west front wall did not move outward at all, but stopped moving at 120 ms with a permanent inward deflection of approximately 1/4 in. The Fachwerk wall panel deflected to full gage range of 6 in. approximately 33 ms after blast arrival.

The Fachwerk front wall at Structure No. 3 deflected inward to a maximum of approximately 3.7 in. in 60 ms, followed by weak oscillation around a permanent inward deflection of approximately 1/2 in. Period of oscillation is approximately 120 ms. Deflection of the front masonry cavity wall was approximately 0.10 in. peak and 0.02 in. permanent inward. A weak oscillation, amplitude less than 0.1 in. peak, was detected in the record of the north masonry cavity wall at Structure No. 3.

Deflection gage records from Structure No. 2 are reproduced in Figure 52. Zero time corresponds to blast arrival at the west wall.

Total Pressure Gages

All pilaster gages show a seemingly instantaneous jump in pressure followed by a more gradual rise to a maximum some 5 ms later. At Structure No. 1, the initial step is approximately 14.8 psi while the peak is near 16.4 psi. At Structure No. 2, the corresponding pressures are approximately 5.1 and 7.3 psi and at Structure No. 3, 2.8 and 3.6 psi. Clearing of reflected pressure approximately 20 ms after shock arrival is seen at all structures.

The two additional pilaster gages at Structure No. 3 recorded significantly lower peak pressures, i.e., 3.4 and 3.5 psi, than did the central gage and both records go to zero after some 70 ms.

Pilaster gage results for Structure No. 2 are shown in Figure 53.

55
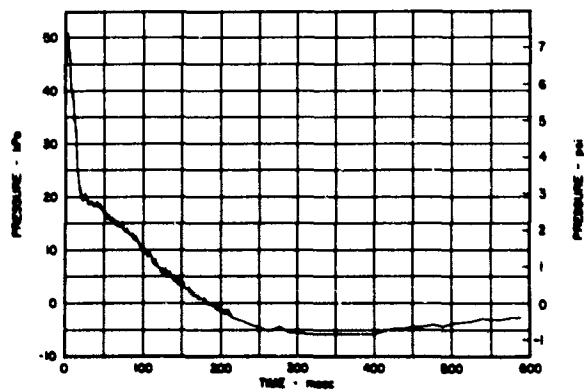
FIGURE 53   TOTAL PRESSURE VERSUS TIME, PILASTER STRUCTURE NO. 2
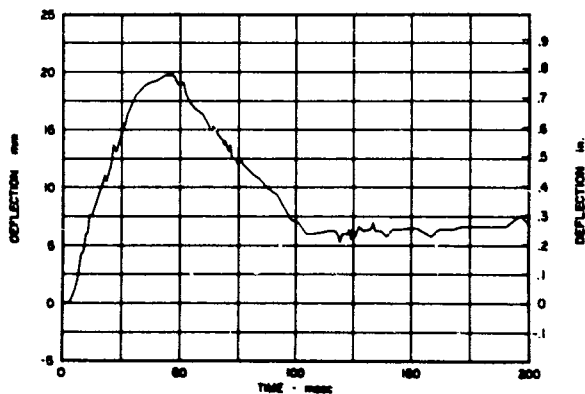
FIGURE 52a   DEFLECTION VERSUS TIME, WEST FRONT
MASONRY CAVITY WALL STRUCTURE NO. 2
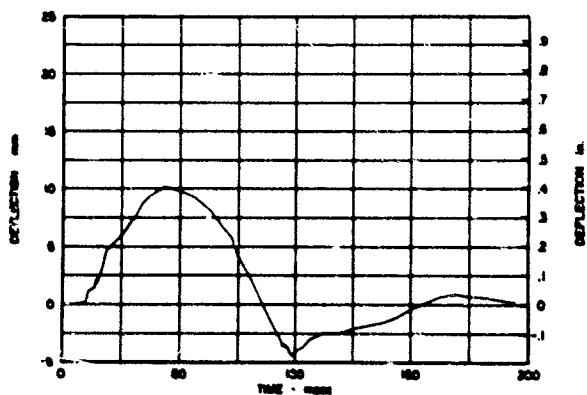
FIGURE 52b   DEFLECTION VERSUS TIME, NORTH SIDE
MASONRY CAVITY WALL STRUCTURE NO. 2
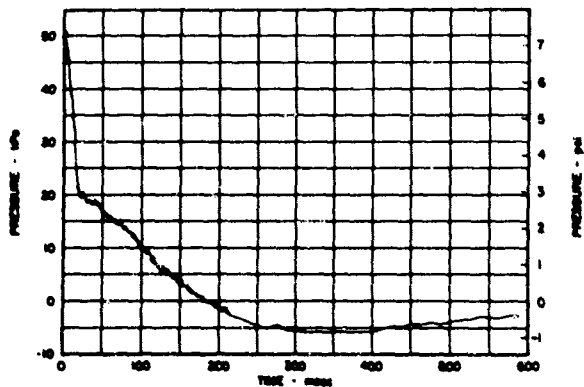
FIGURE 53   TOTAL PRESSURE VERSUS TIME, PILASTER STRUCTURE NO. 2

Side-On Pressure Gages

The ground level gages just outside the north walls all showed an initial step. Unlike the total pressure at the pilaster, this outside side-on pressure showed no significant rise following the discontinuity.

At this time, no attempt has been made to infer shock speed by measuring the time interval between shock arrival at the front wall and at the gage outside the north wall.

All outside side-on pressure gages show a secondary shock arrival approximately 18 ms behind the main shock. The secondary shock, including an accompanying pressure decline, interrupts the normal free-field pressure decay behind the main shock. Normal positive phase pressure is recovered after 30 to 50 ms.

A record from the outside north wall side-on gage at Structure No. 2 is shown in Figure 54. Zero time is shock arrival at the pilaster gage of the same structure.

The interior pressure gages, all installed flush with horizontal surfaces, showed a generally gradual build-up of pressure followed by a decline and negative pressure phase patterned after what is seen in free-field records. Gages inside the Fachwerk room all recorded maximum pressures close to free-field values reached after a 50 to 60 ms rise time. In the masonry cavity wall room maximum pressures were approximately 20 percent lower than in the adjacent Fachwerk room, but rise times were quite similar. Records show some pressure fluctuations which have not been studied yet. In particular, a decline interrupting pressure build-up at the two gages in each structure near windows appears to be associated with vortex formation at the window edge. There is a gradual intensification of this effect when comparing the pressure record from Structure No. 3, where it is weak, to Structure No. 1, where it is strong. Relatively large irregularities in the rising interior pressures at the two interior gages remote from windows of Structure No. 1 are seen also; these irregularities appear much weaker in the other two structures.

The differential pressures across the midpoint of the ceiling slabs show downward pressure approximately twice the upward at all structures.

Pressure traces from the interior gages at Structure No. 2 appear in Figure 55. Zero time in all figures corresponds to shock arrival at the pilaster gage.
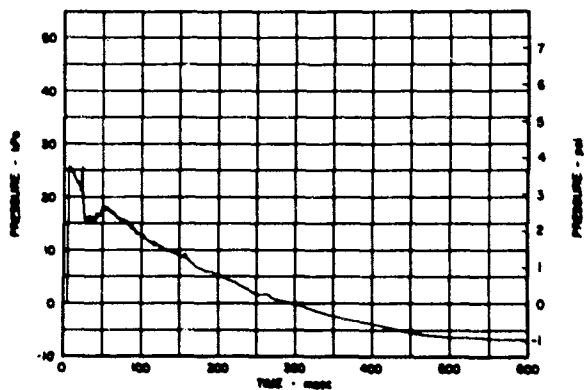
FIGURE 54 PRESSURE VERSUS TIME, OUTSIDE NORTH
SIDE WALL STRUCTURE NO. 2
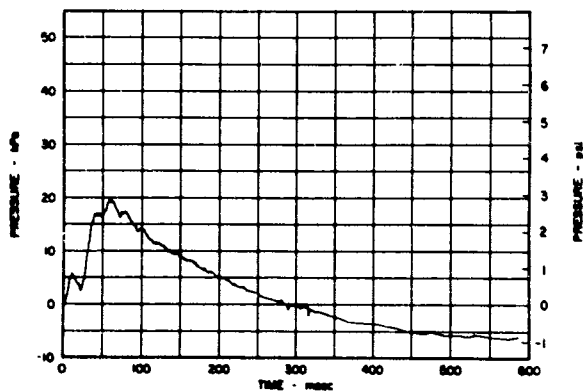
FIGURE 55a   PRESSURE VERSUS TIME, FACHWERK
              ROOM BELOW WINDOW STRUCTURE NO. 2
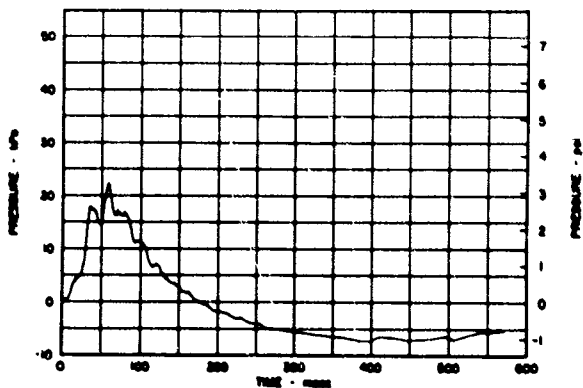
FIGURE 55b   PRESSURE VERSUS TIME, FACHWERK ROOM
              NORTHWEST CORNER STRUCTURE NO. 2
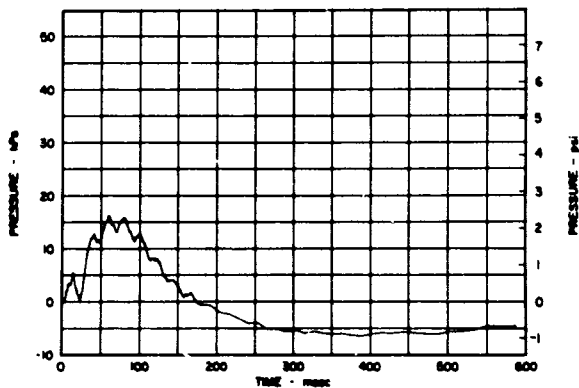
FIGURE 55c   PRESSURE VERSUS TIME, MASONRY CAVITY WALL
              ROOM ABOVE WINDOW STRUCTURE NO. 2
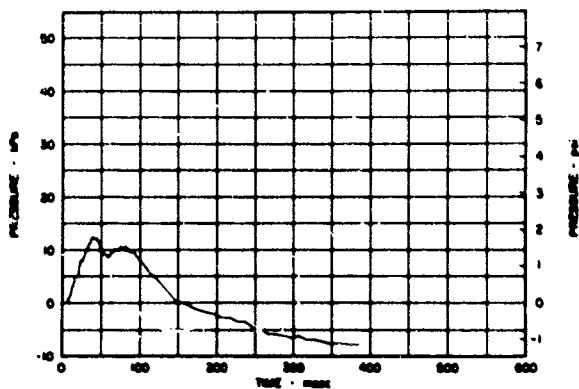
FIGURE 55d   PRESSURE VERSUS TIME, MASONRY CAVITY WALL
              ROOM CENTER OF CEILING STRUCTURE NO. 2

6
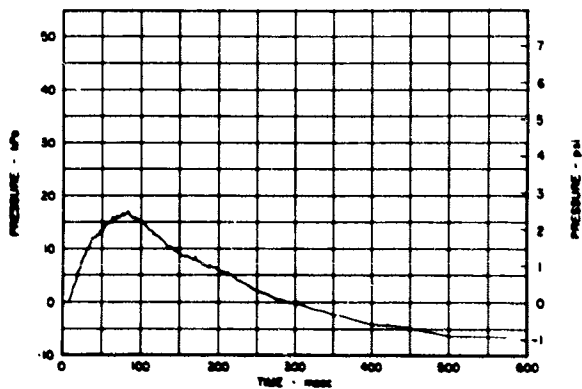
FIGURE 55e   PRESSURE VERSUS TIME, MASONRY CAVITY WALL
ROOM, BESIDE NORTH WALL STRUCTURE NO. 2

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31. TEST SUMMARY - NBDS OPTICAL MEASUREMENTS

by

M. Oberst

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TEST SUMMARY - NBDS OPTICAL MEASUREMENTS

BACKGROUND AND OBJECTIVES

A Nuclear Burst Detection System (NBDS) sensor station
and central processing console (CPC) were fielded at Dice
Throw. The objective of the experiment was to utilize NBDS
equipment to measure the azimuth and time history of an
optical event and thereby demonstrate capabilities of the
NBDS. The equipment used consisted of a tripod mounted sensor
station, a prototype junction box, a hand held display, hard
wired communications, van mounted CPC and a motor-generator
set. The equipment was located at WEAP site, 13,280 feet
from ground zero.

In order to accomplish the stated measurements, certain
modifications were required to prevent system rejection of
the event as non-nuclear and to allow full-time history
recording. The sensor station changes involved elimination
of the EMP coincidence requirement and modification of soft-
ware to transmit the time history of any optical trigger
received. The CPC software was modified to allow receipt
and printing of time history messages and to provide the
capability of a quick-look data plot. Elimination of the
EMP coincidence discriminant introduced a problem of false
triggers whenever the sun entered the field-of-view (FOV)
of the sensor station. This, together with the 8:00 AM
shot time necessitated the use of a sun shade. A 4 x 5 foot
sheet of masonite was erected on a portable tower approximately
33 feet from the sensor station. The center of the shade was
located at 103° true azimuth (EAST from NORTH) and 10°
elevation such that it blocked the sun for a sufficient period
around shot time to allow the optical measurement to be made.

TIME HISTORY MEASUREMENT

Attachment 1 and 2 show the Dice Throw data as printed out
and plotted by the CPC. The amplitude is by level detector
(LD) value and time is by sample number. Since both scales
are only quasi-logarithmic, one simple conversion factor does
not apply. However, when point by point conversions are
applied, the time history as shown in attachment 3 results.
For comparison purposes, attachment 4 is a plot of data taken
at Dice Throw with an older style instrument (not saturation)
and attachment 5 is a plot of data taken with an instrument
fielded at WEAP site as part of a USAFTAC experiment. All
three plots (attachments 3 through 5) compare favorably.

1

AZIMUTH MEASUREMENT

Attachment 6 shows a top view of one of the NBDS sensor station azimuth detectors. The azimuth is measured as the ratio of light received by the azimuth channel to that received by a reference channel. The ratio, encoded as a number between zero and 1023, is a measure of the angle from the channel ambiguity. The ambiguity was set for Dice Throw at 9° south of due west. The true azimuth (EAST from NORTH) of Dice Throw was 55° placing it at a NBDS azimuth of 206°. Since NBDS sensor stations are designed to receive scattered light (FOV +4° to +10°), a correction factor is computed and applied at the CPC. This correction of 12° indicates an expected azimuth reading of 194° (NBDS) or 552 counts. A look at the bottom of attachment 1 shows the actual azimuth reading obtained was 731 counts* or 257° (NBDS). The indicated azimuth error is 257° minus 194° equals 63°. Attachment 6 diagrams and summarizes the various angles.

A post-shot analysis of the NBDS sensor station azimuth capability revealed a sensitivity to light at vertical angles below 4° which was previously considered negligible. At the close range of Dice Throw, the direct light component received at these low vertical angles becomes comparable to the scattered component received from the intended FOV. This direct component tends to shift the measured azimuth toward higher ratios and is sufficient to account for the Dice Throw azimuth error. Unfortunately, the sensitivity of the calculations to certain measured values is such that an exact direct light correction cannot be achieved. Dice Throw, therefore, does not permit a good estimate of NBDS sensor station optical azimuth accuracy.

SUMMARY

The operation of the NBDS sensor station, hand-held display, junction box, CPC and the associated algorithms developed for Dice Throw performed as expected. The objective of measuring optical time history was accomplished. The objective of measuring sensor azimuthal accuracy could not be determined. Unexpected results were obtained due to the sensor sensitivity to the direct light component at vertical angles less than 4°.

*This azimuth is selected from the 12 obtained since it was based upon the highest amplitude signal.

2

TIME..06/10/76..07:59:58

...............TIME HISTORY................

.066.	071.	087.	099.	101.	107.	111.	117.	123.	127.	126.	129.	.0.	131.	131.	132...16
.133.	133.	133.	134.	134.	134.	135.	135.	135.	135.	135.	135.	135.	135.	135.	135*...32
.135.	135.	135.	135.	135.	134.	134.	134.	134.	134.	134.	134.	133.	133.	133.	133...48
.133.	133.	133.	132.	132.	132.	132.	132.	132.	132.	132.	132.	131.	131.	131.	131*...64
.131.	131.	131.	131.	130.	130.	130.	130.	130.	130.	130.	129.	129.	129.	129.	129...80
.129.	129.	129.	129.	128.	128.	128.	129.	128.	128.	128.	128.	128.	128.	128.	128*...96
.128.	128.	127.	127.	127.	127.	127.	127.	127.	126.	126.	126.	126.	125.	125.	124...112
.124.	124.	123.	122.	122.	122.	122.	121.	120.	120.	120.	120.	120.	118.	118.	118*...128
.117.	116.	116.	115.	114.	114.	113.	112.	112.	112.	111.	110.	110.	109.	109.	108...144
.108.	107.	107.	106.	106.	106.	105.	105.	104.	104.	104.	103.	103.	102.	102.	102*...160
.101.	100.	100.	099.	098.	098.	097.	097.	096.	096.	096.	096.	096.	096.	096.	096...176
.093.	092.	088.	085.	084.	081.	080.	078.	077.	076.	074.	073.	072.	071.	070.	070*...192
.049.	067.	066.	064.	076.	074.	072.	065.	060.	056.	053.	048.	044.	042.	040.	040...208
.038.	036.	036.	032.	032.	032.	032.	028.	026.	032.	019.	018.	015.	014.	011.	011*...224
.007.	007.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000...240
.000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000*...256
.000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000...272
.000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000*...288
.000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000*...304
.000.	000.	000.	000.	000.	000.	000.	000.	010.	000.	000.	000.	000.	000.	000.	000*...320
.000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000...336
.000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000*...352
.000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	070.	000.	070*...368
.000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000*...384
.000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000...400
.000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000.	000*...416

AZIMUTH 0-1073
0768. 0872. 2316. 0731. 0748. 0767
0895. 1073. 0000. 1073. 0768. 0000
CHECKSUM ERROR COUNT 000

Attachment 1

PL

DECIMAL LEVEL

DICE THROW

OCT. 6, 1976

TIME IN SECONDS

RELATIVE AMPLITUDE

Attachment 3

5

YSR ND1.0
DICE THROW
630 TON ANFO
13,300 FT RANGE
WSMR OCTOBER 6, 1976

Attachment 4

6

SS)/AC91
*ID 6

Attachment 5

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	RATIO	READING	ANGLE FROM AMBIGUITY	ANGLE FROM NORTH
DUE NORTH	.73	742	261°	0°
ACTUAL READING	.71	731	257°	4°
TRUE DICE THROW	.57	689	206°	55°
EXPECTED READING	.54	552	194°	67°
	.50	512	180°	81°
DUE EAST	.48	486	171°	90°
SYSTEM AMBIGUITY	0 or 1	0 or 1023	0°	261°

Attachment 6

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TEST SUMMARY - NBDS/MAST ASSEMBLY

OBJECTIVE

The purpose of this experiment was to demonstrate structural capability of the NBDS/mast system exposed to a blast environment. Nuclear blast survivability is a part of the NBDS design criteria and Dice Throw provided the best opportunity for evaluating this requirement.

ANALYSIS

Pretest analysis of the NBDS/mast system indicated several problem areas. The analysis was performed with the use of the SHELL-SHOCK computer code. This is a SLA in-house code used for predicting the response of structures to dynamic loads. The code can combine a variety of springs, masses, and shell and solid elements. The mast was divided into 50 shell elements, the NBDS package into 11 masses, and the effect of the guy wires were simulated by linear springs.

The suspected problem areas were the high loads in the guy wires, unsupported mast length, the joint at the base of package, and the optical base.

Early calculations indicated that the three guy wire systems showed excessively high loads in the cables and also an unsupported length of 20' brought the tube close to yielding. We, therefore, recommended a four guy system. The four guy system still indicated high loads in the 1/8" cables and larger diameter cables appeared to be desirable. We were attempting to provide a non-yielding cable system, in order to assure vertical alignment. This meant installing larger ground anchors and cables. The top cables were 5/16" diameter and the others were 1/4" diameter. The intended design should have provided the desired characteristics, except the cable clamps did not perform anywhere near the capacity of the cables. Two cables failed prematurely because of large slippage in the cable clamps. In fact, nearly all the cables showed clamp slippage.

The dynamic analysis also indicated a whipping effect of the NBDS package. A very high moment is induced at the joint and the base plate was modified to sustain this load. Although no fractures were predicted, the yielding would be substantial to cause misalignment of the NBDS package.

The last area of concern was the optical base. The base was made of a laminated glass-phenolic and this type of

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material usually exhibits a large coefficient of variation
(1 σ value). Although failure was not predicted, we did
want to establish this design for a blast environment.

TEST RESULTS

The results of the experiment show we met our intended
objectives. The NBDS base plate and optical support survived
the blast environment. Preliminary pressure traces indicate
that predicted overpressure (5 psi) was achieved. No instru-
mentation was installed in this experiment. Camera coverage
was provided.

The camera was located about 80' from the mast centerline
and provided a field of view of almost the entire mast and
cables (See Figure 1). The films showed blast arrival time
at 660 msec and the predicted time was 660 msec. (Further
times are referenced to structural response times and t = 0
is at shock arrival.) The mast centerline shows about 5"
lateral displacement at 10 msec. Other displacements were
too small for resolution up to 80 msec. At this time, a
forward guy broke at the #3 level and freed the upper center
portion of the mast. A peak displacement occurred at 160-175
msec and was on the order of 26".

The actual displacement was probably larger than this
since there is a degree of freedom away from the camera. Also,
during this time the actual dynamic pressure is very low
(∿ .25 psi). The positive phase duration of the overpressure
lasted for 315 msec compared to a predicted 291 msec.

The tower itself did not fail or yield. In the post-test
configuration, the tower was leaning forward because of the
weight of the heavier cables. When the broken cables were
returned to the anchors, the mast returned to its pre-test
configuration. If the cables had been standard issue, we
suspect the tower would return to the vertical position.

It was noted, that similar communication masts in another
test area all showed cable yielding or stretching as well as
ground anchor displacements 2-3 inches. Because the cables
were light-weight, most of the towers righted themselves.
It would appear that the stronger cable system is not needed
and we recommend that the standard 1/8" cables be continued
as the most desirable design.

10
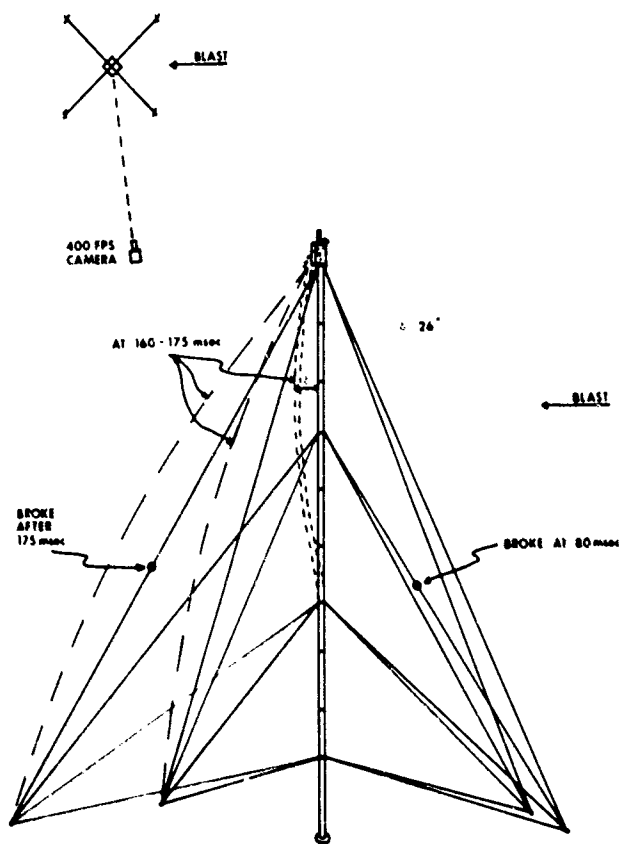
FIGURE 1.   DICE THROW - NBOS/MAST CONFIGURATION

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32. RESULTS OF THE SPECIAL FORCES
COMMUNICATIONS EXPERIMENT
AT THE DICE THROW MAIN EVENT

by

Capt. Alexander F. Wojcicki

SOC, USAOCCS

INTRODUCTION

DICE THROW offered a unique opportunity to gain experimental data on the operational characteristics of Special Forces-peculiar radio sets in desert and nuclear blast environments. Additionally, quantitative data on g-loading to failure of certain critical radio components was desired; experience had indicated that the AN/PRC-74B was more prone to failure than the AN/GRC-109 during the conduct of parachute operations.

OBJECTIVES

Overall objectives of the experiment were:

1. To evaluate Special Forces communications techniques in a desert environment.

2. To evaluate the response of low-powered amplitude-modulated (AM) radio sets and associated antennas and power supplies to air blast and shock at low overpressures (1.0 psi predicted).

3. To evaluate shock response of critical components at selected higher overpressures.

CONDUCT AND RESULTS OF THE LOW OVERPRESSURE EXPERIMENT

1. At the low overpressure site, a variety of antenna/radio combinations were evaluated using authorized frequencies in the 2.000 to 15.000 MHz range. Termination point for all radio traffic was the 7th Special Forces Group Operational Base at Fort Bragg, North Carolina. Two contact a day were planned, with continuous operation from T-120 minutes to T-45 minutes and from T+.75 minutes to T+60 minutes. The following antennas and radios were tested:

a. Antennas: ½-wave di-pole, inverted 'L', inverted 'V', slant wire, and multiple wave long wire.

b. AN/GRC-109, G43 generator, manual morse (CW) mode.

c. AN/GRC-109, G43 generator, AN/GRA-71 burst code device mode.

d. AN/GRC-109, .125 kW generator, manual CW mode.

e. AN/GRC-109, .125 kW generator, AN/GRA-71 burst code device mode.

f. AN/PRC-74B, BA-4386, manual morse mode.

g. AN/PRC-74B, BA-4386, AN/GRA-71 burst code device mode.

2. Data to be obtained included: determination of most effective system, measurement of dB degradation of signal transmitted through the burst cloud,

:

measurement of gross deformation of antenna masts or breakage of antennas,
detection of visible damage to radio sets, and determination of GO/NO GO
condition of each radio and antenna.

3. Radio operators were on site at the time of detonation to transmit
message traffic immediately after detonation and to evaluate the systems.
Seventeen personnel, including control and safety personnel, were on the
test bed at time of detonation. Permission for personnel to remain on-site
was obtained from the Test Group Director, once written approval from The
Office of the Surgeon General was granted.

4. Results:

a. Actual overpressures at the experiment site were 1.3-1.5 psi for
exposed personnel and equipment, rather than the 1.0 psi predicted. Over-
pressure values are taken from BRL gauge line -1 at 1522 m/5000 ft.

b. Communications circuits between the test bed and the Special Forces
Operational Base were not established during the experiment period. It is
believed that the cause was low frequency propagation probabilities for
the assigned frequencies at the time of detonation.

c. No damage to any portion of the antenna array, to any of the radio
components, or to any of the power supplies was noted after detonation.

d. No injuries of any kind occurred among personnel on-site. No one
noticed a sensation of ringing in the ears after detonation (all personnel
wore over-the-ear hearing protectors.)

CONDUCT AND RESULTS OF THE HIGH OVERPRESSURE EXPERIMENT

1. The high overpressure phase of the experiment was conducted by BRL
in the C^3 area. One AN/PRC-74B and one T-784 transmitter (AN/GRC-109) were
placed on ground at the 7.3 psi predicted overpressure level; one T-784
transmitter at the 5.0 psi predicted level; and one AN/PRC-74B at the
3.0 psi predicted level. It was believed that the equipment at the 7.3 psi
level would fail, thereby providing a proven 'gate', or overpressure/g-load-
ing factor above and below failure. Instrumentation included wired accelero-
meters attached at the longitudinal plane center of mass. Accelerometers were
uni-axial, and oriented horizontally, perpendicular to the plane tangent to
the incident shock wave.

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2. Results:

a. Overpressure levels were less than predicted. None of the radios suffered visible damage and all were operational post-detonation. Consequently, only those components at the highest overpressure levels were analyzed.

b. AN/PRC-74B (did not fail)

    (1) Static overpressure........................ 6.0 psi

    (2) Dynamic overpressure........................ .98 psi

    (3) System mass/surface area exposed........... .01 kg/cm^2

    (4) Translation distance........................ 63.5 cm

    (5) Maximum delta-g............................ 840 g

    (6) Maximum delta-g/delta-t.................... 655/.45 (g/ms)

c. T-784

    (1) Static overpressure........................ 6.0 psi

    (2) Dynamic overpressure........................ .98 psi

    (3) System mass/surface area exposed........... .13 kg/cm^2

    (4) Maximum delta-g............................ 386 g

    (5) Maximum delta-g/delta-t.................... 386/.50 (g/ms)

CONCLUSIONS

1. Effective communications in a desert environment are difficult to attain with the AN/PRC-74B and AN/GRC-109 radio sets.

2. Present generation Special Forces communications equipment is able to withstand air blast and shock at a level of 1.5 psi without damage. Critical components can withstand 6.0 psi without damage.

3. At a given overpressure level, the AN/PRC-74B exhibits a much greater shock response than the T-784 transmitter.
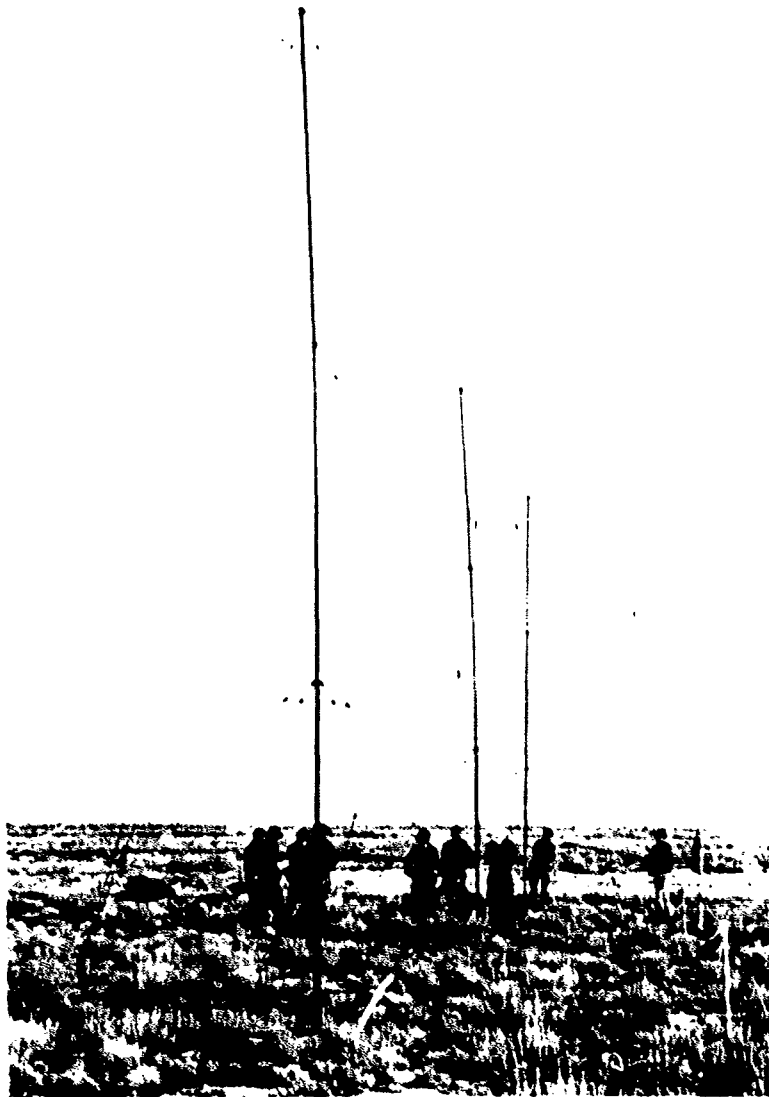
3

Figure 1. Post detonation view of 1.5 psi experiment site.

4

Figure 2. Post-detonation operation of AN/PRC-74B radio set with AN/GRA-71 burst code device.

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33. CRATER AND EJECTA ENHANCEMENT STUDIES

by

R.W. Henny

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